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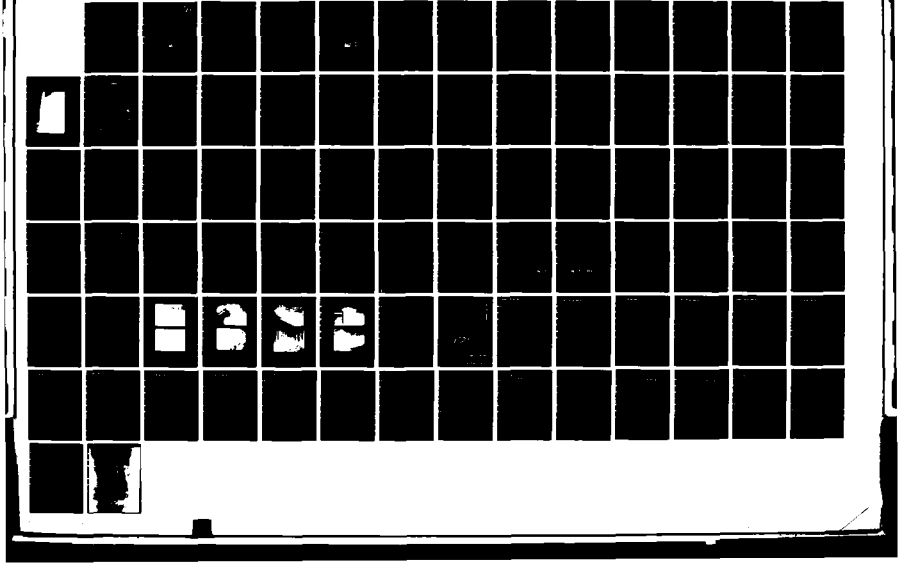

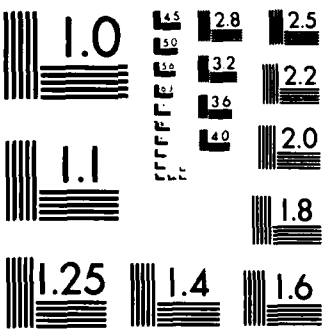

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HOUSATONIC RIVER BASIN
WOLCOTT, CONNECTICUT

CHESTNUT HILL DAM
CT 00298

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

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	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle): Chestnut Hill Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS	5. TYPE OF REPORT & PERIOD COVERED: INSPECTION REPORT
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION	8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS: DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254	12. REPORT DATE: June 1981 13. NUMBER OF PAGES: 65
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18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,
Housatonic River Basin
Wolcott, Connecticut

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Chestnut Hill Dam is 36.1 ft. high and consists of an earth embankment with a concrete corewall and has a total length of 788 ft., including a 28 ft. long concrete spillway with permanent stop planks at the left end. It is classified as a high hazard, intermediate size dam. The test flood for the project is equivalent to the Probable Maximum Flood. Based upon the visual inspection at the sit and past performance, the project is judged to be in fair condition.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:

NEDED

SEP 10 1981

Honorable William A. O'Neill
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor O'Neill:

Inclosed is a copy of the Chestnut Hill Dam (CT-00298) Phase I Inspection Report, prepared under the National Program for Inspection of Non-Federal Dams. This report is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. I approve the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is vitally important.

Copies of this report have been forwarded to the Department of Environmental Protection, and to the owner, Scovill, Inc., Waterbury, CT. Copies will be available to the public in thirty days.

I wish to thank you and the Department of Environmental Protection for your cooperation in this program.

Sincerely,

C. E. EDGAR, III
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

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HOUSATONIC RIVER BASIN
WOLCOTT, CONNECTICUT

CHESTNUT HILL DAM
CT 00298

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name:	CHESTNUT HILL DAM
Inventory Number:	CT 00298
State:	CONNECTICUT
County:	NEW HAVEN
Town:	WOLCOTT
Stream:	OLD TANNERY BROOK
Owner:	SCOVILL INCORPORATED
Date of Inspection:	APRIL 22, 1981
Inspection Team:	PETER M. HEYNEN, P.E.
	HECTOR MORENO, P.E.
	THEODORE STEVENS
	RICHARD LAURIA

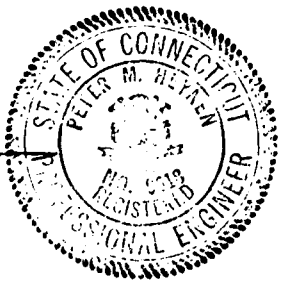
Chestnut Hill Dam was built around 1919 to replace an earlier dam on the site. It presently impounds a recreational reservoir as well as providing watershed storage for downstream concerns. The 36.1 foot high dam consists of an earth embankment with a concrete corewall and has a total length of 788 feet, including a 28 foot long concrete spillway with permanent stop planks at the left end. The embankment has a top width of 13 feet with a 2 horizontal to 1 vertical upstream slope protected by hand placed riprap to within 2 feet of the top and a 2 horizontal to 1 vertical downstream slope with a 7 foot wide berm about 12 feet from the top. A concrete gate chamber and brick gatehouse are located on the upstream side of the embankment. The gatehouse houses two hand wheel pedestal gate lifts controlling a 12 inch outlet and a 20 inch outlet from the gate chamber. Both outlets connect to a 20 inch cast iron low-level outlet pipe. With the reservoir level to the top of the project, the dam impounds approximately 2,000 acre-feet of water.

In accordance with U.S. Army Corps of Engineers guidelines, Chestnut Hill Dam is classified as a high hazard, intermediate size dam. The test flood for the project is equivalent to the Probable Maximum Flood (PMF). Peak inflow to the reservoir at test flood is 3,300 cubic feet per second (cfs); peak outflow is 2,060 cfs with the low point at the left end of the embankment overtopped by 1.1 feet. Peak inflow at the 1/2 PMF is 1,650 cfs and peak outflow is 780 cfs, with the reservoir level to elevation 641.0 and the dam maintaining 0.6 feet of freeboard. The spillway capacity with the reservoir level to the top of the dam is 980 cfs, which is equivalent to 48% of the routed test flood outflow.

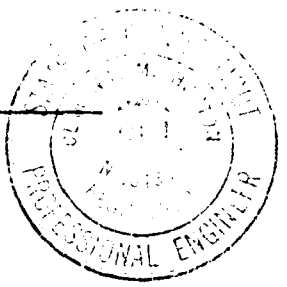
Based upon the visual inspection at the site and past performance, the project is judged to be in fair condition. No evidence of instability of the project was observed. There are items which require attention, such as trees and brush on and at the toe of the embankment, seepage through the embankment, deterioration of the concrete spillway training wall and outlet headwall, the poor alignment of the spillway discharge channel and low-level outlet channel, and the unknown condition of the 12 inch outlet from the gate chamber.

It is recommended that the owner retain the services of a registered professional engineer to perform a more detailed hydraulic/hydrologic investigation to assess further the potential for overtopping of the dam and the need for, as well as the means to increase project discharge capacity. Other items of importance are investigation and monitoring of seepage, removal of trees, repair of the spillway training wall and low-level outlet headwall, renovation of the spillway discharge channel and low-level outlet channel, and investigation of the operability of the 12 inch gate chamber outlet.

The above recommendations and further remedial measures presented in Section 7 should be instituted within one year of the owner's receipt of this report.



Peter M. Heynen, P.E.
Project Manager - Geotechnical
Cahn Engineers, Inc.



C. Michael Horton, P.E.
Chief Engineer
Cahn Engineers, Inc.

This Phase I Inspection Report on **Chestnut Hill Dam (CT 00298)** has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgement and practice, and is hereby submitted for approval.

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

JOSEPH W. FINEGAN, JR., CHAIRMAN
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

The information contained in this report is based on the limited investigation described above and is not warranted to indicate the actual condition of the dam. The integrity of the dam can only be determined by a means of a monitoring program and/or a detailed physical investigation. The accuracy of available data is assumed where not in obvious conflict with facts observable during the visual inspection.

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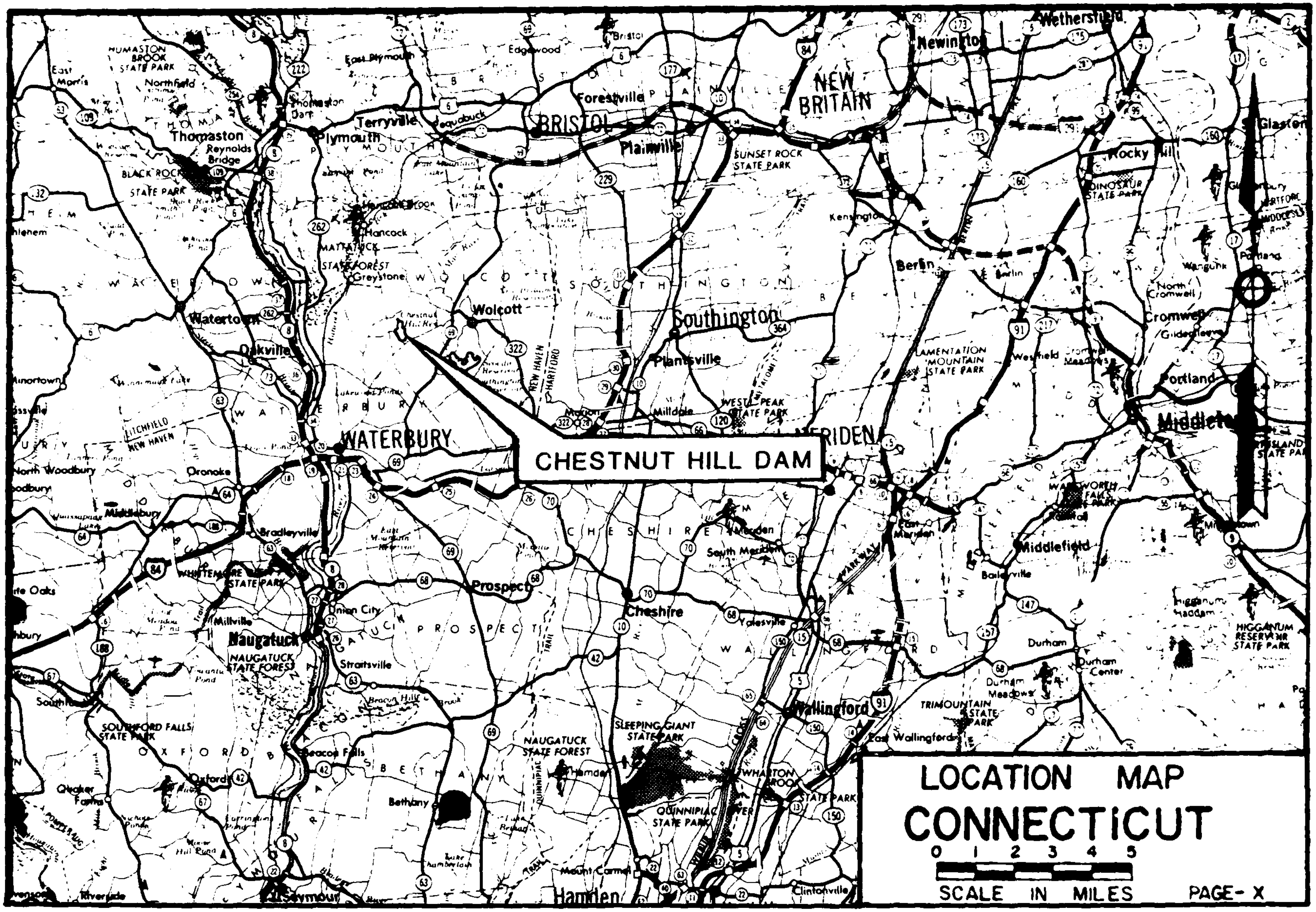
APPENDICES

OVERVIEW PHOTO
(4/22/81)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	Chestnut Hill Dam Old Tannery Brook	Wolcott CONNECTICUT	DATE June, 81 CE #27 785 KI PAGE ix


CHESTNUT HILL DAM
WATERBURY
BRISTOL
NEW BRITAIN
Southington
Wolcott
Plainville
Cheshire
Prospect
Naugatuck
Wallingford
Thomaston
Plymouth
Terryville
Watertown
Oakville
Berlin
Middlefield
Durham
Newington
Wethersfield
Hamden
LOCATION MAP
CONNECTICUT
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SCALE IN MILES
PAGE- X

PHASE I INSPECTION REPORT

CHESTNUT HILL DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Rhode Island. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 7, 1981 from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW 33-80-C-0052 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The project is located on Old Tannery Brook, about 2-1/2 miles above the confluence with the Mad River (Housatonic River Basin), in a rural area of the Town of Wolcott, County of New Haven, State of Connecticut and is shown on the Waterbury USGS Quadrangle Map having coordinates of latitude N41°35.7' and longitude W73°00.4'.

b. Description of Dam and Appurtenances - As shown on Sheet B-1, the dam is an earth embankment approximately 788 feet in length and 36.1 feet in height. The top of the embankment is 13 feet wide and the elevation at the crown is 642.3, however within 100 feet from the right spillway training wall, the embankment slopes downward to meet the top of the training wall (elevation 641.5). This area would be the first point of overtopping should the water level exceed elevation 641.6, and is considered in this report to be the top of the dam. The upstream slope is at an inclination of 2 horizontal to 1 vertical and is protected with hand-placed riprap to elevation 640. The downstream slope is at an inclination of 2 horizontal to 1 vertical with a 7 foot wide berm at elevation 629.

Design drawings of the project show the present dam to replace an earlier earth dam and to contain a concrete corewall which is 30 feet upstream of the axis of the old dam. Much of the present dam embankment on the downstream side of the corewall is composed of a portion of the old dam that was left in place. The dam is shown to be founded on rock at its left end, on hard gravel in the center and on sandy hardpan near the right end. At its maximum depth, the bottom of the corewall is at elevation of approximately 590, giving the dam a total structural height of approximately 52 feet.

The spillway, located at the left end of the dam, is a 28 foot long concrete weir with 1.5 foot high stop planks, which have a crest elevation of 637.0. The left end of the spillway abuts a concrete end wall with a top elevation of 638.1. From this wall, a grassy side slope extends upward to the left. There is a concrete training wall along the right side of the spillway and, approximately 30 feet downstream of the spillway crest, this training wall abuts a dry-laid stone masonry wall along the right side of the spillway discharge channel. The spillway discharge channel appears to have been cut into natural ground, with the masonry wall to the right and a grassy slope to the left.

As shown on Sheet B-2, the intake structure consists of a brick gatehouse atop a concrete gate chamber, which was constructed on the heavy stone masonry foundation of the old gate chamber. Intake to the chamber is through three approximately 4 foot by 5 foot inlets in the upstream wall of the chamber. These rectangular inlets, with invert elevations of 605.5, 617.75 and 629.5, are designed to be controlled with stop logs slid in grooves in the upstream wall. Discharge from the chamber is through a 20 inch cast iron pipe which is fed by two outlets from the chamber. The two outlets are a 20 inch main line and a 12 inch branch joining in a Y-section of pipe immediately downstream of the gate chamber. Both outlets, at invert elevation 605.5, are regulated by sluice gates which are controlled by hand wheel floor stands in the gatehouse.

The 20 inch cast iron discharge pipe passes through the corewall of the dam and then connects to a 30 inch cast iron pipe left in place from the old dam. This pipe extends to a concrete headwall at the toe of the dam and discharges to a stone paved outlet channel.

c. Size Classification - (INTERMEDIATE) - The dam is 36.1 feet in height and has a storage capacity of 2,000 acre-feet. According to U.S. Army Corps of Engineers Recommended Guidelines, a dam with storage capacity between 1,000 and 50,000 acre-feet is classified as intermediate in size.

d. Hazard Classification - (HIGH) - The potential impact area in case of failure of the dam consists of several homes, apartment buildings, factories and a shopping mall, all located between 7,000 and 14,000 feet downstream of the dam along Old Tannery Brook and Mad River. If the dam were breached, these buildings would be flooded by 1 to 9 feet of water, creating the potential for loss of more than a few lives.

e. Ownership - Scovill Inc.
Corporate Headquarters
Scovill Plaza
500 Chase Parkway
Waterbury, CT 06708
(203) 757-6061
Heminway Merriman
Vice President-Secretary
Mr. Michael Palumba
Plant Manager

The dam has been under the same ownership since its construction.

f. Operator - Century Brass Products Inc.
59 Mill St.
Waterbury, Ct. 06720
(203) 574-7700
Mr. Steve Zainc
Water Analyst
Mr. Tom Carroll
Chief Engineer

g. Purpose of Dam - The dam is used to provide storage so that a steady flow of water for manufacturing purposes is assured at Century's Plant on the Mad River in Waterbury. The reservoir is also used for recreational purposes such as swimming and fishing.

h. Design and Construction History - Working drawings of the project dated 1917 and 1918 indicate that much of the design work for reconstruction of the old dam took place during those years. Two drawings which depict the as-built condition of the dam are dated September 1919, indicating that design was finalized at that time. It is assumed that construction took place shortly thereafter. The design and construction history of the old dam is not known.

i. Normal Operational Procedures - Mr. Zainc of Century Brass reports that he checks the dam on a bi-weekly basis. Water is released through the 20 inch outlet as needed to augment flows in the stream below the dam. The water level is normally maintained at the elevation of the top of the stop planks, especially in the summer for recreational purposes.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area is 1.5 square miles of heavily developed rolling terrain.

b. Discharge at Damsite - Discharge is through the 20 inch cast-iron outlet from the gate chamber and over the spillway.

1. Outlet Works 20 inch low-level outlet @ invert el. 605.5:	70± cfs (reservoir level to top of dam el. 641.6)
2. Maximum flood at damsite:	August 1955 - Discharge unknown
3. Ungated spillway capacity @ top of dam el. 641.6:	980 cfs
4. Ungated spillway capacity @ test flood el. 642.7:	1380 cfs
5. Gated spillway capacity @ normal pool:	N/A
6. Gated spillway capacity @ test flood:	N/A
7. Total spillway capacity @ test flood el. 642.7:	1380 cfs
8. Total project discharge @ top of dam el. 641.6:	1050 cfs
9. Total project discharge @ test flood el. 642.7:	2060 cfs

c. Elevations - All elevations given in this report are on National Geodetic Vertical Datum (NGVD), based on assumed top of stop planks elevation of 637.0 corresponding to the reservoir water surface elevation shown on the USGS Waterbury Quadrangle Map, 1972. On the design drawings of the project, a vertical datum, established in 1917, was used whereby the top of stop planks elevation is given as 601.5. Therefore, there is a conversion of 35.5 feet added to the 1917 elevations to obtain NGVD elevations.

1. Streambed at toe of dam:	605.5
2. Bottom of corewall:	590.0±
3. Maximum tailwater:	Not known
4. Normal pool:	637
5. Full flood control pool:	N/A
6. Spillway crest (ungated)	
Concrete weir crest:	635.5
Top of stop planks:	637.0 (assumed datum)
7. Design surcharge (original design):	Not known
8. Top of Dam:	
Crown:	642.3
First point of overtopping:	641.6
9. Test flood surcharge:	642.7

d. Reservoir Length

1. Normal pool:	3000± ft.
2. Flood control pool:	N/A
3. Spillway crest pool: (top of stop planks)	3000± ft.
4. Top of dam pool:	3100± ft.
5. Test flood pool:	3100± ft.

e. Reservoir Storage

1. Normal pool:	1500± acre-ft.
2. Flood control pool:	N/A
3. Spillway crest pool: (top of stop planks)	1500± acre-ft.
4. Top of dam pool:	2000± acre-ft.
5. Test flood pool:	2050± acre-ft.

f. Reservoir Surface

1. Normal pool:	84± acres
2. Flood control pool:	N/A

3. Spillway crest pool: (top of stop planks)	84± acres
4. Top of dam pool:	94± acres
5. Test flood pool:	94± acres
g. Dam	
1. Type:	Earth embankment
2. Length:	788 ft.
3. Height:	36.1 ft.
4. Top width:	13 ft.
5. Side slopes:	2H to 1V (Upstream and Downstream)
6. Zoning:	Downstream side of embankment consists of portion of old dam left in place
7. Impervious core:	Concrete corewall founded in hard gravel or bedrock
8. Cutoff:	N/A
9. Grout curtain:	N/A
10. Other:	7 ft. wide berm on downstream slope at el. 629
h. Diversion and Regulating Tunnel	N/A
i. Spillway	
1. Type:	Round-crested concrete weir with 1.5 ft. stop planks
2. Length of weir:	28 ft.
3. Crest elevation	
Concrete weir crest:	635.5
Top of stop planks:	637.0
4. Gates:	N/A

5. Upstream channel:	Gently sloping, gravel bottom
6. Downstream channel:	Bedrock and bouldery bottom, steep grassed slope on left, dry-laid masonry wall on right.
7. General:	Concrete end wall to left of spillway with top el. of 638.1 functions as high-stage or auxiliary spillway

j. Regulating Outlets

Low-level outlet

1. Invert:	605.5
2. Size:	20 inch diameter to 30 inch diameter
3. Description:	Cast Iron
4. Control mechanism:	Handwheel floor stand lifts
5. Other:	One 20 inch outlet and one 12 inch outlet, both from the gate chamber, are connected to the 20/30 inch low-level outlet.

SECTION 2: ENGINEERING DATA

2.1 DESIGN DATA

The available data consists of inventory data by the State of Connecticut, the Mad River Company storage data, construction drawings from 1917-1919, and correspondence (See Appendix B).

Two drawings of the project, dated September, 1919, are consistent with the design features observed in the field. Several drawings, dated 1917 and 1918, were made available from the owner and appear to be earlier designs which were slightly revised to the design shown on the September, 1919 drawings.

The existing drawings of the project are on a datum established in 1917 assuming the weir crest of the old dam to be elevation 600.0. All elevations given in this report are based on an assumed top of stop planks elevation of 637.0, corresponding to the NGVD elevation of 637 shown on the USGS Waterbury Quadrangle Map. On the 1917 datum, the dam was designed to have a spillway crest elevation of 600.0 with 1.5 foot high flashboards to elevation 601.5. Therefore, elevation 601.5 (1917 datum) corresponds to elevation 637.0 (NGVD datum) and there is a conversion of 35.5 feet added to the 1917 elevations to obtain NGVD elevations.

2.2 CONSTRUCTION DATA

No information is available

2.3 OPERATIONS DATA

Reservoir level readings are taken bi-weekly at the dam and recorded at the power plant of Century Brass in Waterbury.

2.4 EVALUATION OF DATA

a. Availability - Available data was provided by the State of Connecticut and the owner. The owner and the operator made the project available for visual inspection.

b. Adequacy - Since detailed design drawings are available, the assessment of the project was based on a review of these drawings as well as visual inspection, performance history, hydraulic computations of spillway capacity, and hydrologic estimates.

c. Validity - A comparison of record data and visual observations reveals no significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The project is in fair condition. The inspection revealed several areas requiring maintenance and monitoring. At the time of inspection, the reservoir surface was extremely choppy with water splashing over the stop planks.

b. Dam

Top of Dam - The top of dam is in good condition (Photo 1). Grass cover is good although it is matted in places due to vehicular and/or foot traffic on the dam. The top elevation of the dam is consistent, except in an area within 100 feet from the spillway, where it slopes down from elevation 642.3 to 641.6 to meet the top of the right spillway training wall. The chain link fence along the downstream edge of the top of the dam is generally in good condition, but there are a few places where it has been broken.

Upstream Slope - The upstream slope of the dam is in fair condition. Riprap is in good condition although numerous saplings and small trees are growing from between the stones. Erosion probably due in part to trespassing is evident at a few locations, most notably on either side of the gatehouse where depressions greater than 6 feet across are present (Photo 2).

Downstream Slope - The downstream slope is in poor condition. There is extremely heavy brush growth at several locations, hindering inspection of the slope and trees are present throughout the slope (Photo 3). The entire toe is generally wet and soft. A 4 inch cast iron pipe, which discharges to the low-level outlet channel, appears to be the outlet for a toe drain, installed to correct the wet area at the right side of the embankment (Photo 4). The outlet of the pipe is partially submerged below the water in the outlet channel, inhibiting free flow from the pipe. Flow from the pipe is estimated at 2 to 3 gallons per minute (gpm) and was not observed to be transporting any soil. Seepage from the wet area at the left side of the embankment runs along the toe to the low-level outlet channel, about 5 feet below the outlet headwall. The total rate of seepage from the left side of the dam is estimated to be 3-4 gpm, however no pipe exists on this side.

Spillway - The spillway is in fair condition (Photo 5). The concrete spillway crest is in good condition with very little deterioration of the concrete; however, the concrete training wall is spalled. The stop planks are in good condition. The design drawings of the project show the toe of the side slope to the left of the dam meeting the top of the spillway end wall approximately 15 feet from the edge of the spillway; however, it was observed that the slope actually meets the end wall only 3.8 feet from the edge of the spillway. The auxiliary spillway (end wall) appears to have been partially covered by placement of fill (Photo 5), thereby reducing the combined spillway capacity.

The spillway discharge channel is poorly defined, is vegetated with trees and brush, and contains much debris (Photo 6). The dry-laid masonry wall along the right side of the channel, probably dating from the original dam, is in good condition as is the abutment between this wall and the concrete spillway training wall.

c. Appurtenant Structures - The low-level outlet facilities are in fair condition. The 20 inch outlet from the gate chamber is well-lubricated and operable, but the operator reports that he has never operated the 12 inch outlet. Although the operability of the 12 inch outlet gate is untested, the handwheel floor stand for this outlet appears to be in good condition. The portion of the concrete gate chamber visible above the water surface appears to be in good condition, with only minor spalling. The presence or absence of stop logs in the slots for the gate chamber inlets could not be determined and there was no mechanism in the gate house for the placement or removal of stop logs in the slots. The brick gatehouse appears to be in sound condition (Photo 7). The low-level outlet headwall is in poor condition, exhibiting extensive spalling of concrete (Photo 8). The headwall was also observed to be wet. The spalling and wetness of the concrete may be a result of seepage through the dam in the area of the headwall and possibly through the headwall itself. The lower portion of the outlet channel is filled with debris, blocking outlet discharge and partially submerging the low-level and toe drain outlets with 1.5 feet of water.

d. Reservoir Area - There is a paved road along a portion of the right shoreline of the reservoir and two beach clubs near the upstream end of the reservoir. There are 3 apartment buildings at the top of the side slope to the left of the spillway and a sewer line, which was built in 1978-79 and runs along the left side of the reservoir and spillway channel. It is possible that placement of additional fill on the concrete end wall of the spillway occured during construction of the sewer line.

e. Downstream Channel - Both the spillway channel and the low-level outlet channel discharge to a shallow pond approximately 150 feet from the toe of the dam. This pond discharges through two 30 inch concrete pipes under a road approximately 300 feet downstream of the dam.

3.2 EVALUATION

Based upon the visual inspection, the project is assessed as being in fair condition. The manner in which the features identified in Section 3.1 could affect the future condition and/or stability of the project is as follows:

1. Trees on the upstream slope of the dam, if not removed and allowed to increase in size, could cause displacement of riprap and provide seepage paths through the embankment along root systems.

2. Eroded areas of the upstream slope will continue to erode due to lack of ground cover and/or riprap in some locations.

3. Trees growing on the downstream slope of the dam could provide seepage paths through the embankment or become uprooted, causing damage to the slope.

4. Uncontrolled seepage through the dam has caused wet and/or soft areas at the toe of the dam, is difficult to monitor, and could cause internal erosion of the dam or deterioration of the low-level outlet headwall.

5. Standing water in the low-level outlet channel inhibits inspection of the low-level and toe drain outlet pipes and could be inhibiting free flow from the apparent toe drain discharge pipe, reducing the effectiveness of the toe drain.

6. Continued spalling of the right spillway training wall could decrease the stability of the wall.

7. The fill which has been placed over the spillway end wall reduces the total discharge capacity of the project.

8. The trees, brush and debris in the spillway discharge channel and trees growing from the channel wall will reduce spillway capacity and increase deterioration of the wall.

9. The operability of the 12 inch outlet from the gate chamber is not known, should it be needed as a backup to the 20 inch outlet.

10. It could not be determined whether any abandoned stop logs are blocking the lowermost chamber intake, which would prevent full drawdown of the reservoir, should the need occur.

11. Further deterioration of the low-level outlet headwall could reduce its stability.

12. Extensive brush and tree growth on the embankment prevents thorough inspection of the toe and slopes of the dam.

SECTION 4: OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 OPERATIONAL PROCEDURES

a. General - Water level readings are taken on a bi-weekly basis. The reservoir level is normally kept at the top of stop planks elevation to provide storage for recreational purposes. Water may be released from the reservoir through the 20/30 inch low-level outlet to provide flow downstream. Water is also released during periods of high precipitation or unusually high spring inflows.

b. Description of Any Warning System in Effect - No formal downstream warning system is in effect.

4.2 MAINTENANCE PROCEDURES

a. General - There is no formal program for maintenance at the dam, however the dam is checked for vandalism on a bi-weekly basis and the chain link fence along the top of the dam is repaired on an as-needed basis. The gatehouse windows were blocked up several years ago in an effort to prevent vandals from entering the gatehouse. When necessitated by vandalism, the gatehouse is repaired.

b. Operating Facilities - The 20 inch outlet gate stand is exercised and lubricated on a regular basis, but no maintenance is performed on the 12 inch outlet gate stand.

4.3 EVALUATION

The maintenance procedures are poor. A formal program of maintenance procedures should be implemented, including documentation to provide records for future reference. Remedial maintenance procedures are presented in Section 7.3.

SECTION 5: EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 GENERAL

The Chestnut Hill Dam watershed is 1.5 square miles of rolling terrain.

The dam is an earth embankment with a concrete spillway. The available storage reduces the outflow from a Probable Maximum Flood (PMF) of 3300 cubic feet per second (cfs) to 2060 cfs and the ½ PMF outflow from 1650 cfs to 780 cfs.

5.2 DESIGN DATA

No computations could be found for the original design of the dam.

5.3 EXPERIENCE DATA

The operator reports that the maximum depth of flow over the stop planks that he has observed is only a few inches.

During the winter of 1978-79 when the sewer line to the left of the spillway was being constructed, the reservoir water level was lowered to 180 inches below the top of stop planks on December 30, 1978. Distances below the top of stop planks recorded during January 1979 were 120" on January 9, 81" on January 22, 40" on January 25, 26" on January 27, and 13" on February 3.

5.4 TEST FLOOD ANALYSIS

Based upon the U.S. Army Corps of Engineers "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978; the watershed classification (Rolling), and the watershed area of 1.5 square miles, a PMF of 3300 cfs or 2200 cfs per square mile is estimated at the damsite. In accordance with the size (intermediate) and hazard (high) classification, the test flood is selected as equivalent to the PMF. The reservoir level at the start of the test flood is considered to be at top of stop planks elevation 637.0. Peak inflow to the reservoir at the test flood is 3,300 cfs and peak outflow is 2,060 cfs, increasing the reservoir level to 1.1 feet above the first point of overtopping at the left end of the embankment. Based on hydraulic computations, the spillway capacity to the first point of overtopping (elevation 641.6) is 980 cfs which is equivalent to 48% of the routed test flood outflow (Appendix D-6).

The peak inflow at 1/2 PMF conditions is 1,650 cfs and peak outflow is 780 cfs with the reservoir level increasing to elevation 641.0, leaving 0.6 feet of freeboard to the first point of overtopping.

5.5 DAM FAILURE ANALYSIS

Many houses and industrial/commercial structures with first floors ranging from 6 to 14 feet above the stream constitute the potential impact area in case of failure of Chestnut Hill Dam. These are located along the Old Tannery Brook and the Mad River between 7000 and 14,000 feet downstream from Chestnut Hill Reservoir. In particular, at least five houses with first floors between 8 feet and 13 feet; and five industrial/commercial structures with first floors between 6 feet and 12 feet above the stream are located along Old Tannery Brook between 7,000 and 10,000 feet downstream from the dam. Along the Mad River, between 12,000 and 14,000 feet below the dam (within 2,000 feet from the confluence with Old Tannery Brook), there are two apartment complexes, factories and a large shopping mall with first floors between 11 and 14 feet above the stream; however, the dam failure analysis indicated that a failure of the dam would have little to no effect on these structures.

The dam failure analysis is based on the April, 1978 Army Corps of Engineers "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs". With the reservoir level at the first point of overtopping of the dam, peak outflow before failure would be about 980 cfs and the peak failure outflow from the dam breaching would total about 63,000 cfs.

The prefailure depth of flow at the initial impact area would be 3.8 feet, or approximately 4.2 feet below the first floor of the lowest house (2.2 feet below the lowest structure) in the initial impact area. A breach of the dam would result in a rapid 11 to 12 foot increase in water levels at Old Tannery Brook and 10 feet at Mad River to depths of approximately 15 to 16 feet at the brook and 14 feet at the river. This sudden outflow will cause rapid flooding of several houses by up to 8 feet and of other structures by up to 9 feet, possibly causing loss of more than a few lives and substantial economic loss. Based on the dam failure analysis, Chestnut Hill Dam is classified as a high hazard dam (Appendix D-10).

SECTION 6: EVALUATION OF STRUCTURAL STABILITY

6.1 VISUAL OBSERVATIONS

The visual inspection did not reveal any indications of immediate stability problems. Items described in Section 3, such as trees and brush on the embankment and in the spillway channel, seepage through the embankment, and deterioration of the concrete spillway walls and low-level outlet headwall should be repaired but are not considered to be immediate stability concerns at the present time.

6.2 DESIGN AND CONSTRUCTION DATA

Existing drawings of the project are reproduced as Sheets B-1 & B-2. The drawings indicate that the dam has a structural height of 52 feet, which is approximately 14 feet greater than its hydraulic height; i.e., the lowest footing of the concrete corewall is approximately 15 feet below the low-level outlet channel at the toe of the dam. The corewall is shown to be founded on bedrock near its left end, on hard gravel in the center, and an sandy hardpan near its right end. Considering the gravel soil on which a good portion of the dam is founded, the substantial seepage through the dam is not a cause for immediate concern.

The dam design incorporates portions of the old dam which the present dam replaced. Much of the present dam embankment on the downstream side of the corewall is composed of a portion of the old dam that was left in place, the 20 inch low-level outlet connects to the old 30 inch low-level outlet from the old dam, the concrete gate chamber rests upon the heavy stone masonry foundation of the old gate chamber, and the right side of the spillway discharge channel is lined by a dry-laid masonry wall probably dating from the old dam construction.

None of the design features identified are indicative of a structurally unstable design.

6.3 POST-CONSTRUCTION CHANGES

The 4 inch cast iron pipe discharging to the low-level outlet channel, apparently from a toe drain, is not shown on any design drawings and may have been added later. Construction of the apartment buildings and/or the sewer line at the left end of the dam may have resulted in additional fill being placed on the spillway end wall. While neither of these changes appears to have an adverse affect on the stability of the structure, placement of fill on the end wall appears to reduce the discharge capacity of the project and increases the potential for overtopping.

6.4 SEISMIC STABILITY

The dam is in Seismic Zone 1, and according to U.S. Army Corps of Engineers Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 PROJECT ASSESSMENT

a. Condition - Based upon the visual inspection at the site and past performance, the project is in fair condition. No evidence of instability was observed in the spillway, embankment or appurtenant structures; however, there are several items which require maintenance, repair and monitoring.

Peak inflow to the reservoir at the test flood is 3,300 cfs and peak outflow is 2,060 cfs, with the low point of the dam overtopped by 1.1 feet. The spillway capacity to the top of the dam (elevation 641.6) is 980 cfs, which is equivalent to 48% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the project must be based on visual inspection, past performance, a review of design drawings, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within one year of the owner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that further studies, pertaining to the following items, be conducted by a registered professional engineer qualified in dam design and inspection. Recommendations made by the engineer should be implemented by the owner.

1. A detailed hydraulic/hydrologic investigation to assess further the potential for overtopping of the dam and the need, as well as the means, to provide increased project discharge capacity. This investigation should include an analysis of the adequacy of the spillway discharge channel.

2. Removal of all trees from the dam and from within 15 feet of the toe of the dam. This should include removal of root systems and proper backfilling.

3. Investigation of the significance of the seepage through the dam and establishment of a seepage monitoring program.

4. Investigation of the effectiveness of the apparent toe drain, and if deemed necessary, recommendations for its repair or replacement.

5. Procedures for repair of the right spillway training wall, renovation of the dry-laid masonry discharge channel wall, and removal of trees and rubble from the spillway discharge channel.

6. Procedures for repair of the low-level outlet headwall and renovation of, and removal of debris from the outlet discharge channel.

7. Investigation of the condition of the 12 inch outlet gate from the gate chamber.

8. Inspection of the submerged areas of the concrete gate chamber, including an investigation of the presence or absence of stop logs in the gate chamber intake slots.

7.3 REMEDIAL MEASURES

The following operation and maintenance procedures should be undertaken by the owner and continued on an regular basis.

1. Round-the-clock surveillance should be provided during periods of heavy precipitation or high project discharge. A formal downstream warning system should be developed, to be used in case of emergencies at the dam.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. A comprehensive program of inspection by a registered professional engineer qualified in dam inspection should be instituted on an annual basis.

4. Eroded areas of the upstream slope should be filled, regraded and seeded, with riprap replaced where required.

5. All brush should be removed from the dam and from within 15 feet of the toe of the dam.

7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Chestnut Hill Dam

DATE: 4-22-81

TIME: 2:00 pm

WEATHER: Fair 55°

W.S. ELEV. ____ U.S. ____ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Theodore J. Stevens	TJS	Geotechnical
2. Peter M. Heynen	PMH	Geotechnical
3. Hector Moreno	HM	Hydraulics
4.		
5.		
6.		

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Dam Embankment	All	Fair Cond.
2.	Spillway	All	Fair Cond.
3.	Gatehouse	All	Fair Cond.
4.	Low-level outlet	All	Fair Cond.
5.			
6.			
7.			
8.			
9.			
10.			
11.			
12.			

PERIODIC INSPECTION CHECK LIST

PROJECT Chestnut Hill Dam DATE 4-22-81

PROJECT FEATURE Spillway BY TJS

AREA EVALUATED	CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a) Approach Channel	
General Condition	Good
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	None observed
Floor of Approach Channel	Sand & Gravel
b) Weir and Training Walls	
General Condition of Concrete	Fair
Rust or Staining	None observed
Spalling	Yes - on right training wall
Any Visible Reinforcing	None observed
Any Seepage or Efflorescence	None observed
Drain Holes	None observed
c) Discharge Channel	
General Condition	Poor
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	Yes
Floor of Channel	Bedrock
Other Obstructions	Channel poorly defined, also there is considerable debris in channel

PERIODIC INSPECTION CHECK LIST

PROJECT Chestnut Hill Dam DATE 4-22-81

PROJECT FEATURE Spillway BY TJS

AREA EVALUATED	CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a) Approach Channel	
General Condition	Good
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	None observed
Floor of Approach Channel	Sand & Gravel
b) Weir and Training Walls	
General Condition of Concrete	Fair
Rust or Staining	None observed
Spalling	Yes - on right training wall
Any Visible Reinforcing	None observed
Any Seepage or Efflorescence	None observed
Drain Holes	None observed
c) Discharge Channel	
General Condition	Poor
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	Yes
Floor of Channel	Bedrock
Other Obstructions	Channel poorly defined, also there is considerable debris in channel

PERIODIC INSPECTION CHECK LIST

PROJECT Chestnut Hill Dam DATE 4-22-81

PROJECT FEATURE Gatehouse BY TJS

AREA EVALUATED		CONDITION
OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE		
a) Approach Channel		Approach channel submerged Could not observe
Slope Conditions		
Bottom Conditions		
Rock Slides or Falls		
Log Boom		
Debris		
Condition of Concrete Lining		
Drains or Weep Holes		
b) Intake Structure		Gatehouse
Condition of Concrete		Appears good
Stop Logs and Slots		None in place

PERIODIC INSPECTION CHECK LIST

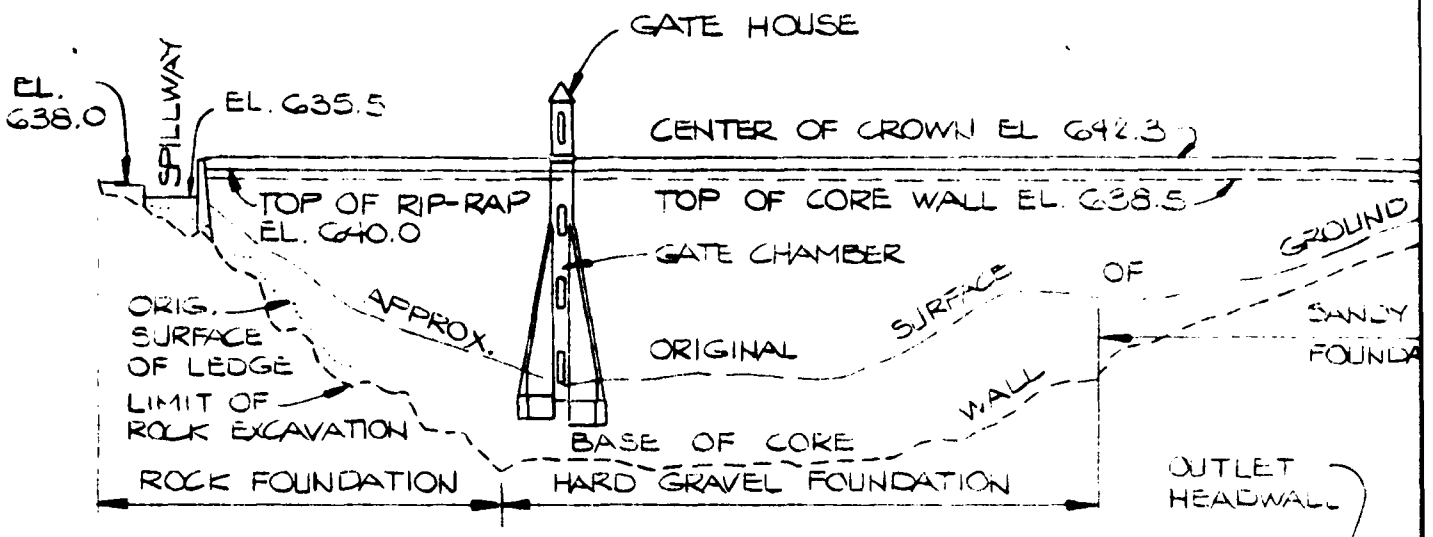
PROJECT Chestnut Hill Dam DATE 4-22-81

PROJECT FEATURE Low-level outlet BY TJS

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		
General Condition of Concrete		Poor
Rust or Staining		None observed
Spalling		Yes -
Erosion or Cavitation		Extensive erosion
Visible Reinforcing		None observed
Any Seepage or Efflorescence		Seepage
Condition at Joints		N/A
Drain Holes		None observed
Channel		
Loose Rock or Trees Overhanging Channel		Thick brush
Condition of Discharge Channel		Poor

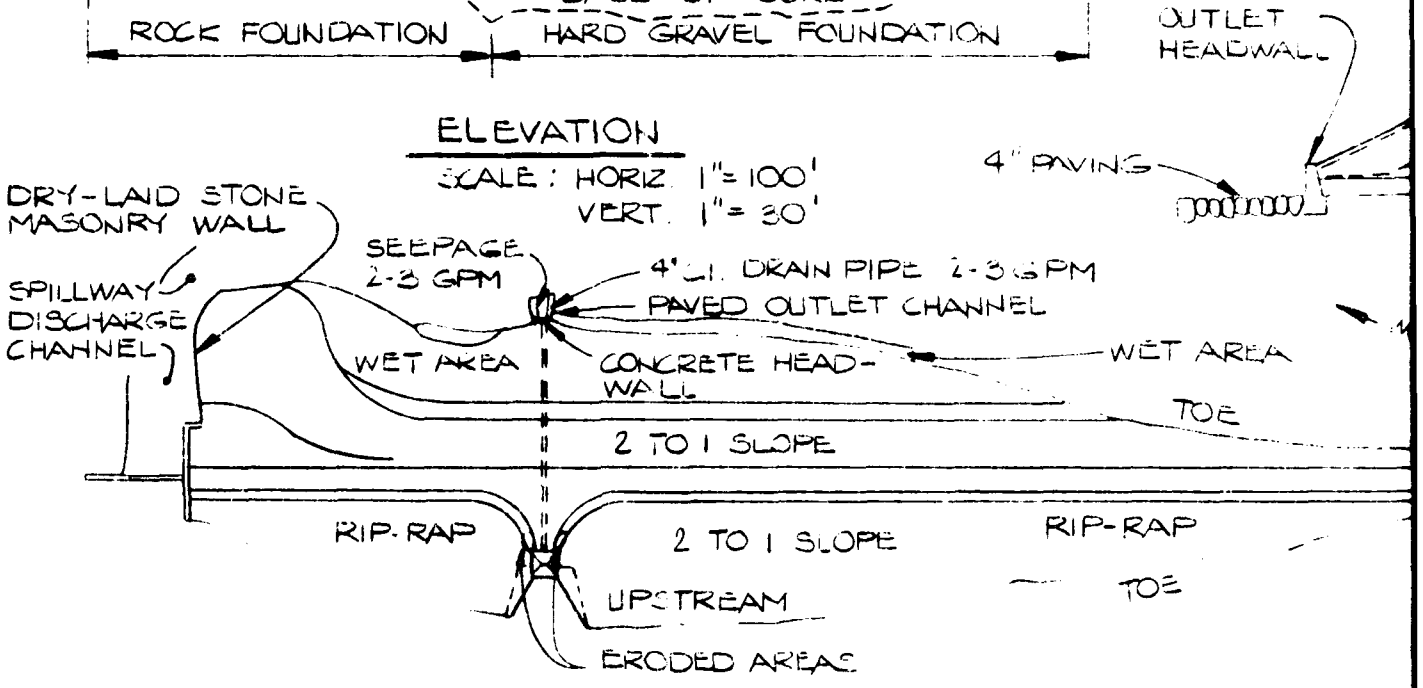
APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE



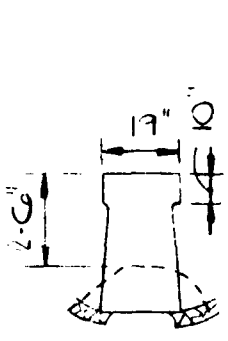
ELEVATION

SCALE: HORIZ. 1"= 100'
VERT. 1"= 30'



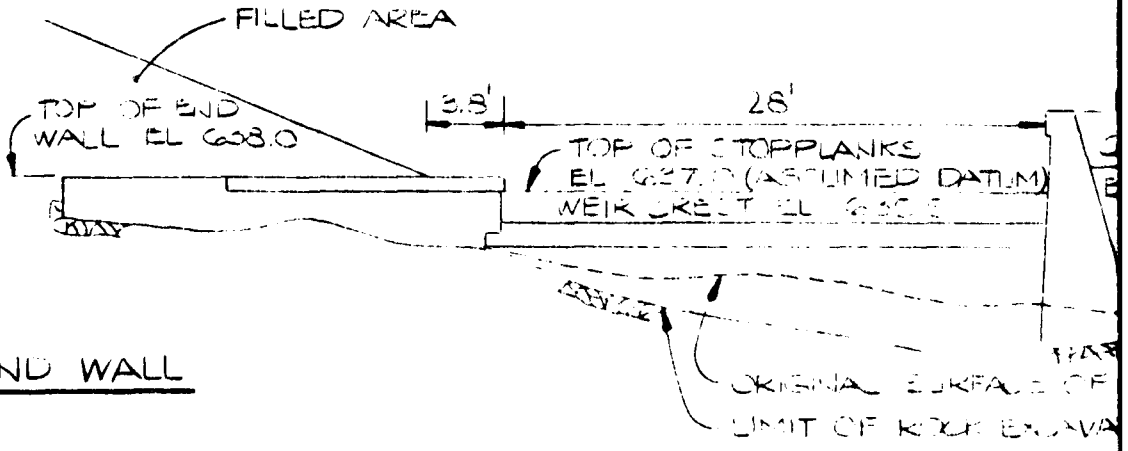
PLAN

SCALE: 1"= 100'



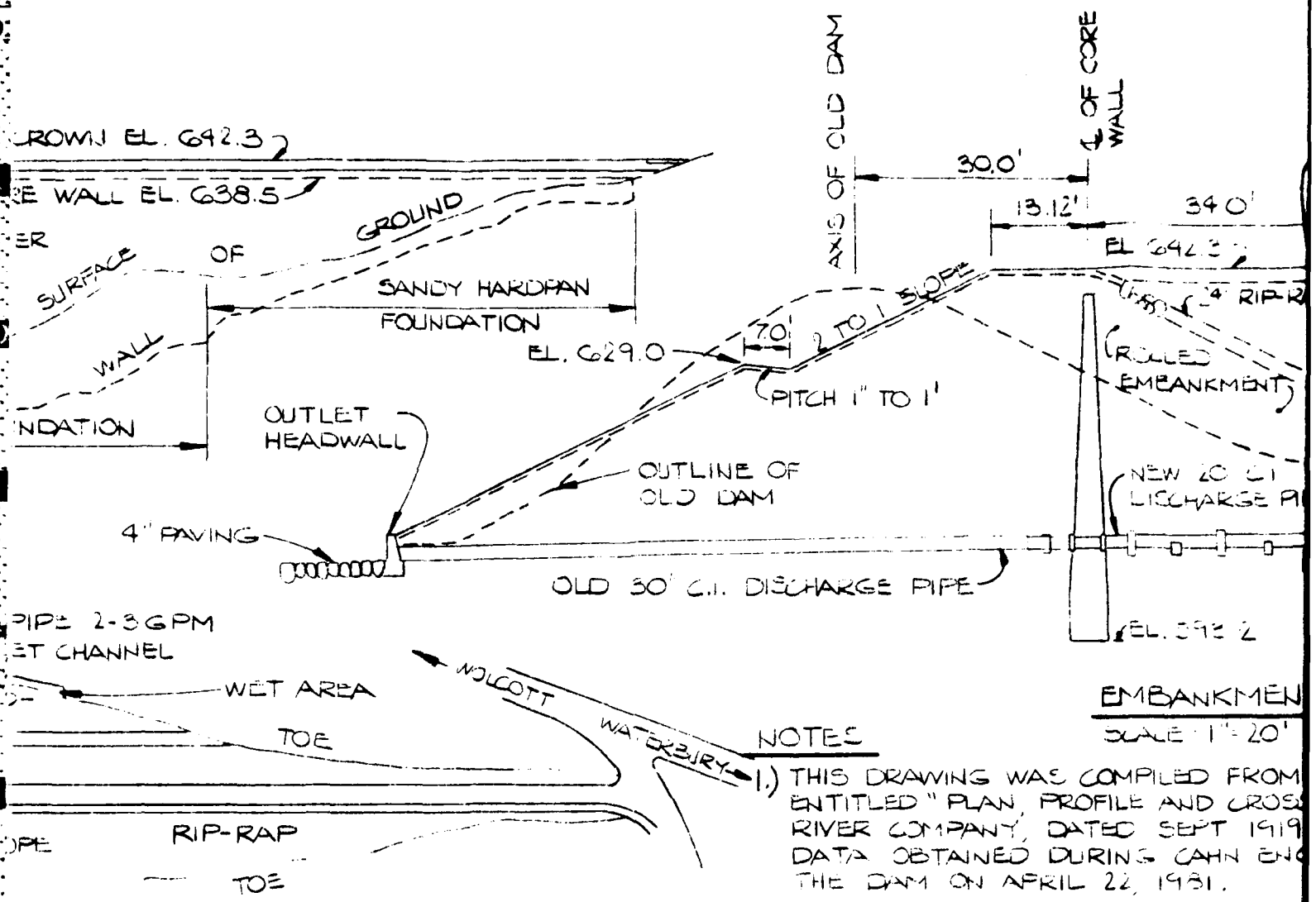
SECTION OF END WALL

SCALE: 1"= 5'



UPSTREAM ELEVATION OF SPILLWAY

SCALE 1"= 10'



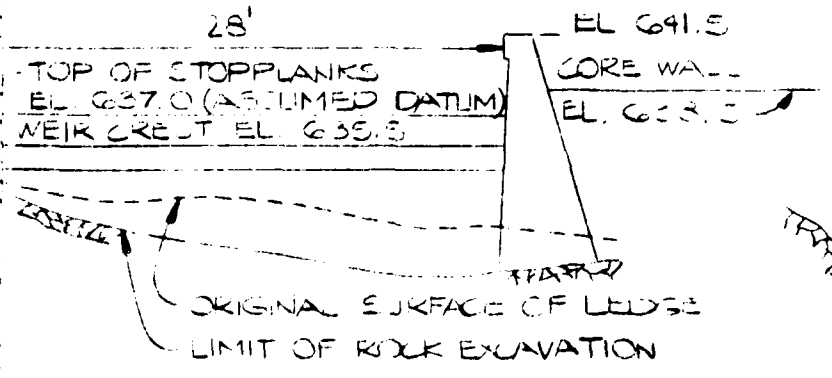
EMBANKMEN

SCALE 1" = 20'

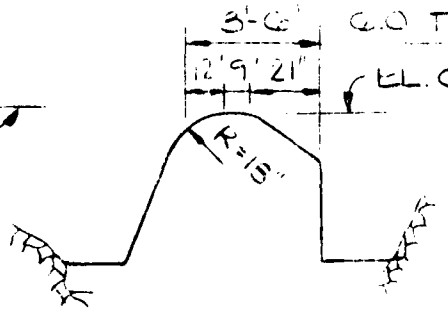
NOTES

1.) THIS DRAWING WAS COMPILED FROM ENTITLED "PLAN, PROFILE AND CROSS RIVER COMPANY", DATED SEPT 1919 DATA OBTAINED DURING CAHN EN THE DAM ON APRIL 22, 1931.

2.) ELEVATIONS ARE N.G.V.D. BASED OF STOPPLANKS ELEVATION OF THE RESERVOIR WATER SURFACE WATERBURY QUADRANGLE MAP. SHOWN ARE REFERENCED TO TH STOPPLANKS ELEVATION.



ELEVATION OF SPILLWAY



SECTION OF SPILLWAY

SCALE 1" = 5'

JOHN ENGINE
WATERFORD, C
N.Y.

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OLD TANNE

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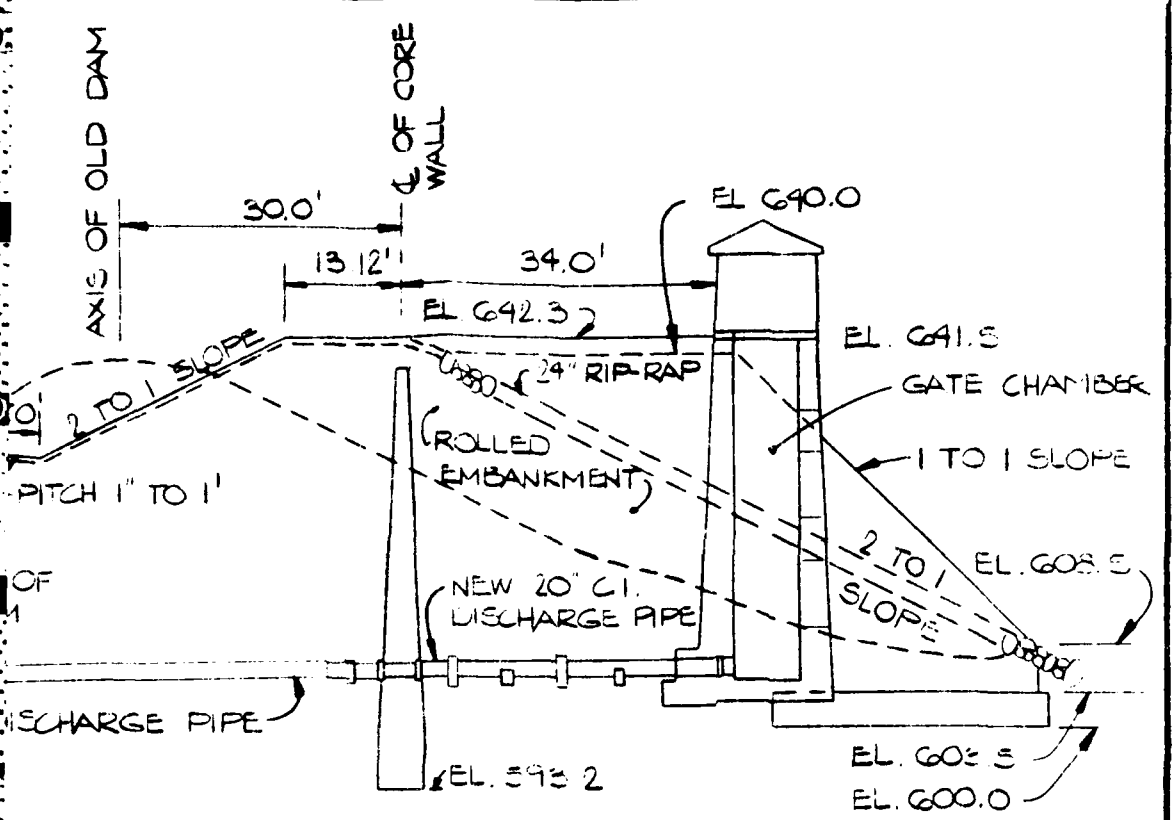


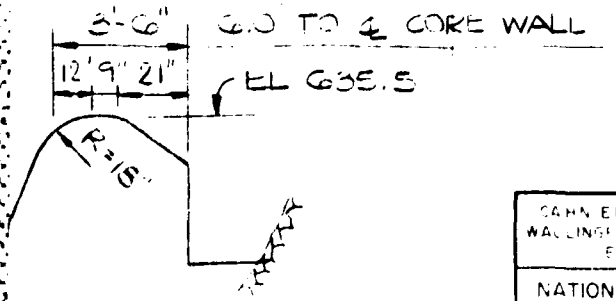
EMBANKMENT SECTION

SCALE: 1"=20'

NOTES

1) THIS DRAWING WAS COMPILED FROM A DRAWING OF THE DAM ENTITLED "PLAN, PROFILE AND CROSS SECTIONS, BY THE MAD RIVER COMPANY, DATED SEPT. 1919 AND SUPPLEMENTAL DATA OBTAINED DURING CAHN ENGINEERS INSPECTION OF THE DAM ON APRIL 22, 1981.

2) ELEVATIONS ARE N.G.V.D. BASED ON AN ASSUMED TOP OF STOPPLANKS ELEVATION OF 637.0 CORRESPONDING TO THE RESERVOIR WATER SURFACE SHOWN ON THE USGS WATERBURY QUADRANGLE MAP. ALL OTHER ELEVATIONS SHOWN ARE REFERENCED TO THE ASSUMED TOP OF STOPPLANKS ELEVATION.



TION OF SPILLWAY

E: 1"=3'

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS

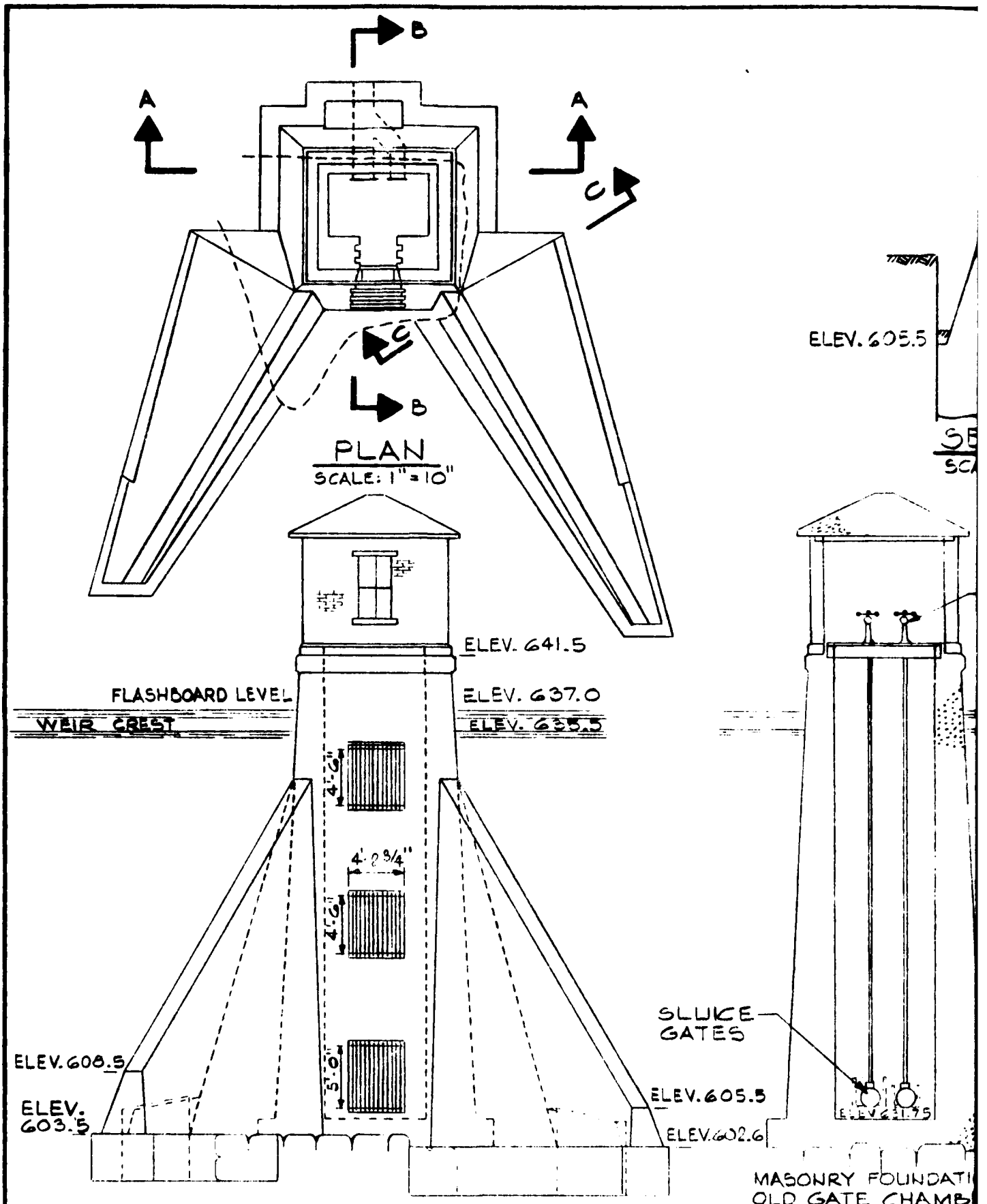
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

PLAN, PROFILE AND SECTIONS

CHESTNUT HILL DAM

OLD TANNERY BROOK WOLCOTT, CONNECTICUT

DWN BY	CKD BY	APP BY	SCALE AS SHOWN	
V.J.J.	[illegible]	[illegible]	DATE JUNE, 1981	SHEET 8 of [illegible]

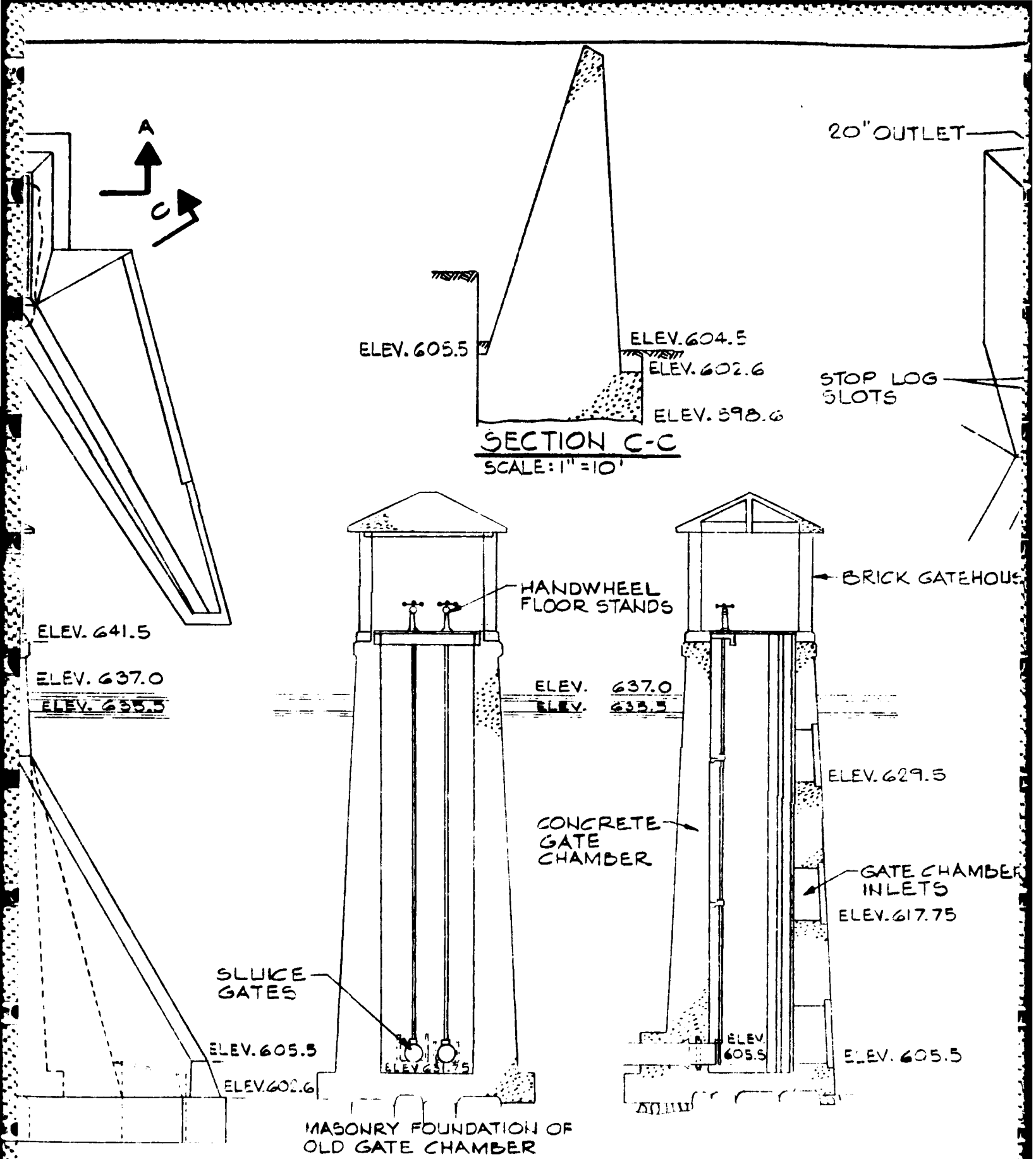


PLAN
SCALE: 1" = 10"

ELEVATION
SCALE: 1" = 10'

MASONRY FOUNDATI
OLD GATE CHAMB

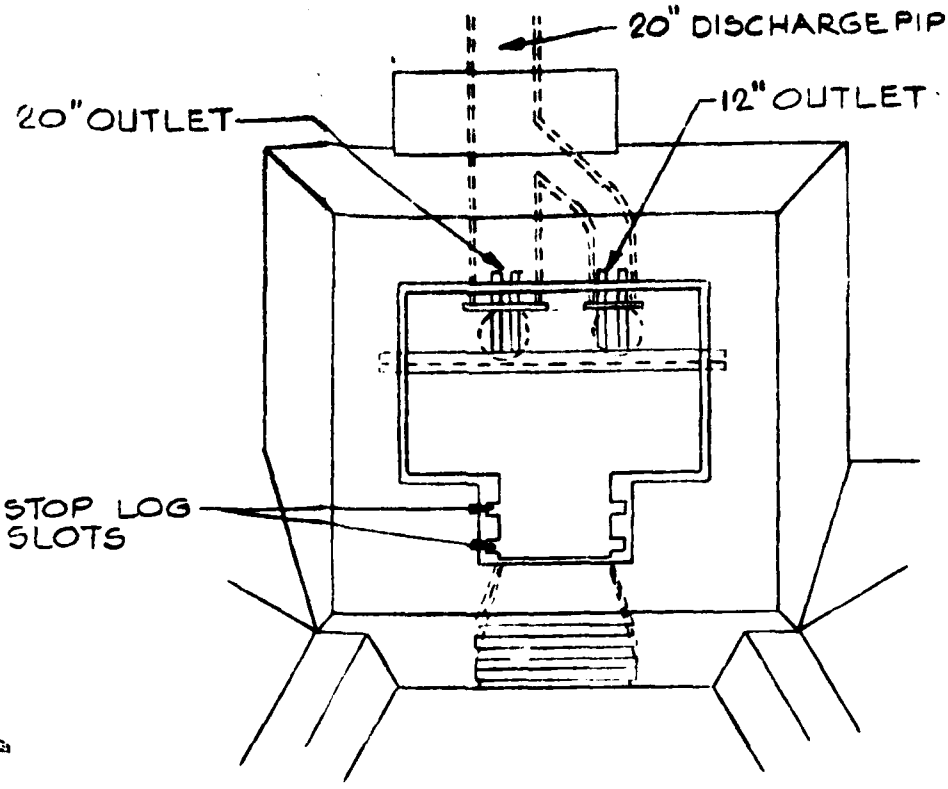
SECTION A
SCALE: 1' = 10'



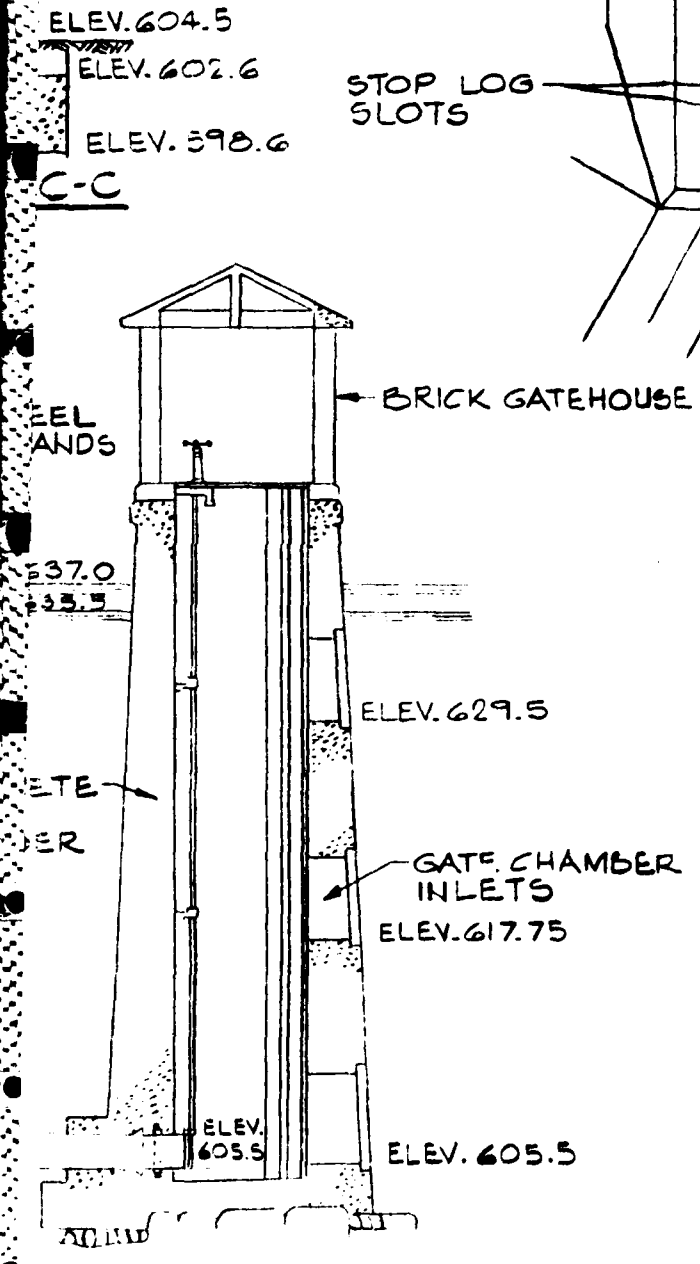
SECTION C-C
SCALE: 1" = 10'

SECTION A-A
SCALE: 1" = 10'

SECTION B-B
SCALE: 1" = 10'



PLAN
SCALE: 1" = 5'



SECTION B-B
SCALE: 1" = 10'

NOTES:

1. THIS DRAWING WAS COMPILED FROM A DRAWING OF THE DAM ENTITLED "GATE CHAMBER" BY THE MAD RIVER COMPANY, DATED SEPT. 1919.
2. ELEVATIONS ARE N.G.V.D. (SEE SHEET B-1, NOTE 2).

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

GATE CHAMBER

CHESTNUT HILL DAM

OLD TANNERY BROOK WOLCOTT, CONNECTICUT

DWN. BY	CKD. BY	APP. BY	SCALE AS NOTED	
F.N.	TJS	PmH	DATE JUNE, 1981	SHEET B-2

CHESTNUT HILL DAM

EXISTING PLANS

Chestnut Hill Dam
Plan, Profile and Cross Section
The Mad River Co.
Waterbury, Conn.
Sept. 1919

Chestnut Hill Dam
Gate Chamber
The Mad River Co.
Waterbury, Conn.
Sept. 1919

Note: The above drawings appear to be substantially consistent with the as-built conditions observed in the field. Earlier drawings of the project, dated 1917 and 1918 were made available by the owner; however, these do not appear to depict the actual as-built conditions of the project.

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
-	Files	State Board for the Supervision of Dams	Inventory Data	B-3
Jan., 1970	Files	The Mad River Co.	Reservoir Storage Data	B-4
May 21, 1942	Vincent B. Clark Supervisor of Dams Ansonia, CT	L.P. Sperry Executive Vice President Scovill Manufacturing Co. Waterbury, CT.	Request to install 24" flashboards	B-5
Jan. 27, 1972	John Luchs John J. Mozzochi & Assoc. Glastonbury, CT	William H. O'Brien, III Civil Engineer Water & Related Resources	Request for inspection of dam	B-6
Feb. 23, 1972	William H. O'Brien, III	John Luchs	Flood routing study	B-7

STATE BOARD FOR THE CONSERVATION OF [illegible]
INVENTORY DATA

CT-297

NAME OF DAM OR POND Chestnut Hill Dam.

CODE NO. N[illegible]

LOCATION OF STRUCTURE:

Town Wolcott

Name of Stream Chittenden Brook Tributary of Mad River

U.S.G.S. Quad. Waterbury Long. 73-00.1 Lat. [illegible]

OWNER: Scovill Manufacturing Company

Address Waterbury

Telephone

6/28/73 1.5 FT splash board; [illegible]

Pond Used For: Recreation DA 1.55sq.m

Dimensions of Pond: Width 1/4 Mi. Length 1/2 Mi. Area 60. A

Depth of Water below Spillway Level (Downstream) 3'

Total Length of Dam ~~750'~~ 630' Length of Spillway ~~50'~~ 19'

187'

Height of Abutments above Spillway ~~2~~ 5'

Type of Spillway Construction Conc.

Type of Dike Construction Earth

Downstream Conditions Brook Flows to Mad River.

Summary of File Data

Remarks Because of size, should be turned over to Board Member.

1/50

THE MAD RIVER CO.

RESERVOIR	WATERSHED AREA	ELEVATION		AREA				
		Weir Crest	Flash Boards	Weir Crest		Flash Boards		Wei
	Sq mi.	Feet	Feet	Sq ft	Acres	Sq ft	Acres	Cu.
Chestnut Hill	1.50	$634.0	635 5	33[illegible]	77 82	3648023	83.75	.5974
Woodtick	8.69	525.0	526 25	5609187	128 77	6038845	138 63	.723
Hitchcock's	0.69	620.0		2217856	50 91			.1637
"	0.33	623.3		2674380	6, 40			2004
Cedar Swamp	1.27	8400		6060353	139 13			.7522
Total	10.88			1995[illegible]2	[illegible]	2063950?	473.82	1929

• Approximate elevation above mean sea level. $ Same as assur

Total capacity with Chestnut Hill and Woodtick at flash-bo level.

Total capacity with Woodtick at flash board and Chestnut Hill, level = 200.43 mill. cu. ft. or 1499.34 mill gals

STORAGE RESERV

AREA			CAPACITY				MAX. DEPTH		MEAN DEPTH	
r	Flash	Boards	Weir	Crest	Flash	Boards	Weir Crest	Flash Boards	Weir Crest	Flash Boards
s	Sq ft	Acres	Cu. ft	Mill gals	Cu ft	Mill gals	Feet	Feet	Feet	Feet
2	3648023	83.75	59748139	446.95	65046107	486.58	316	331	1763	17.83
77	6038895	138.63	72344445	541.34	54741499	409.50	273	2855	8.42	9.06
			16378087	122.52			24		7.38	
2			20040881	14992			11		749	
3			49522571	370.45			163		8.17	
2	20639507	473.82	192924099	1443.17	205730121	1538.17				

[5] Same is assumed elev 600.0 on all maps, plans and cross-sect

dtick of flash-board, and Hitchcock's and Cedar Swamp at we

and Chestnut Hill, Hitchcocks and Cedar Swamp at weir crest

No 872

B-

STORAGE RESERVOIRS.

		MAX. DEPTH		MEAN DEPTH		% MEAN TO MAX. DEPTH	
ash	Boards	Weir Crest	Flash Boards	Weir Crest	Flash Boards		
f^t	M. gals	Feet	Feet	Feet	Feet	Weir Crest	Flash Boards
246107	486.58	31.6	33.1	17.63	17.83	55.8	53.9
741499	409.50	27.3	28.55	8.42	9.06	30.8	31.7
		24		7.38		59.5	
		11		7.49		67.5	
		13.1		8.17		62.4	
730121	1538.17						

o on all maps, plans and cross-sections.

hcocks and Cedar Swamp at weir crest

and Cedar Swamp at weir crest

Nº 872 D6.

SCOVILL MANUFACTURING COMPANY

ESTABLISHED 1802

WATERBURY, CONNECTICUT

EXECUTIVE OFFICES

May 21, 1942

Mr. Vincent B. Clark
Supervisor of Dams
Ansonia, Connecticut

Dear Mr. Clark:

We own the Chestnut Hill Reservoir in Wolcott. Due to water activities, we are using an extraordinary amount of water and last summer were a good deal worried about our supply This year we may use more.

We believe that probably the dam at Chestnut Hill would safely stand 24" flashboards, which might store 30 million gallons more than is retained by the present dam with its present 9" flashboards.

We should like to have your permission to use 24" flashboards or as much as you think would be safe, and we would of course like to get them on soon. Will you be good enough to communicate with Mr. Hugh L. Thompson, our engineer, who can give you all the necessary information with respect to the dam.

Very truly yours,

SCOVILL MANUFACTURING COMPANY

LPS/GD

Executive Vice President

Water & Related Resources

January 27, 1972

Mr. John Luche
John J. Mozzochi & Associates
Consulting Engineers
217 Hebron Avenue
Glastonbury, Connecticut 06033

Re: Chestnut Hill Reservoir Dam
Wolcott

Dear John:

Under the terms of your contract to act as a consultant to this department, would you inspect and send us a report on the safety of the subject dam.

Very truly yours,

William H. O'Brien, III
Civil Engineer

WHO:ljg

cc: Stephen C. Thomson, Director

MOZZOCHI ASSOCIATES
CIVIL ENGINEERS

GLASTONBURY, CONN. 06033
217 HEBRON AVENUE
PHONE 633-9401

PROVIDENCE, R. I. 02903
188 WEYBOSSET STREET
PHONE 421-0420

PARTNERS
JOHN LUCHS, JR
STUART J. BECKERMAN

February 23, 1972

REPLY TO: Glastonbury

William H. O'Brien, III - Civil Engineer
Department of Environmental Protection
Water & Related Resources
State Office Building
Hartford, Connecticut 06115

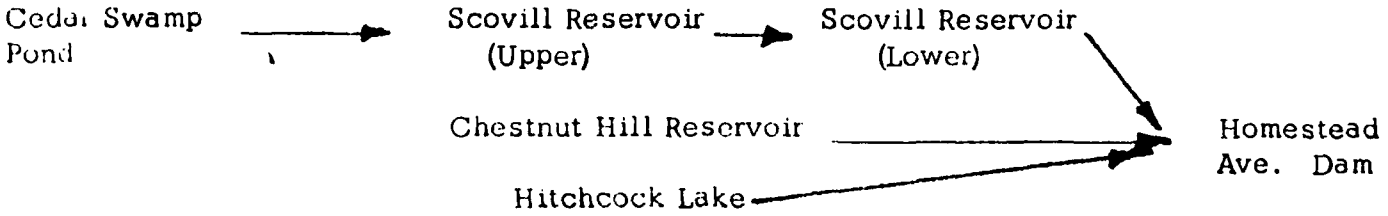
WATER & RELATED RESOURCES
RECEIVED
FEB 25 1972
ANSWERED
REFERRED
FILED

Re: Homestead Ave. Dam
(John Errichetti Assoc.)
Waterbury
Our File No. 57-73-94

Dear Mr. O'Brien:

As authorized in your letter of January 27, 1972, we have inspected and evaluated the spillway capacity allowing for the storage of four large reservoirs in the up-stream drainage area.

The total watershed area for this structure is 17.4 ± sq. miles, with four (4) major reservoirs upstream. The storages and releases from these independent structures, will affect the inflow of the subject dam. It was therefore necessary to determine the routed discharges from the upstream reservoirs to evaluate the inflow hydrograph. The flow pattern is as follows:



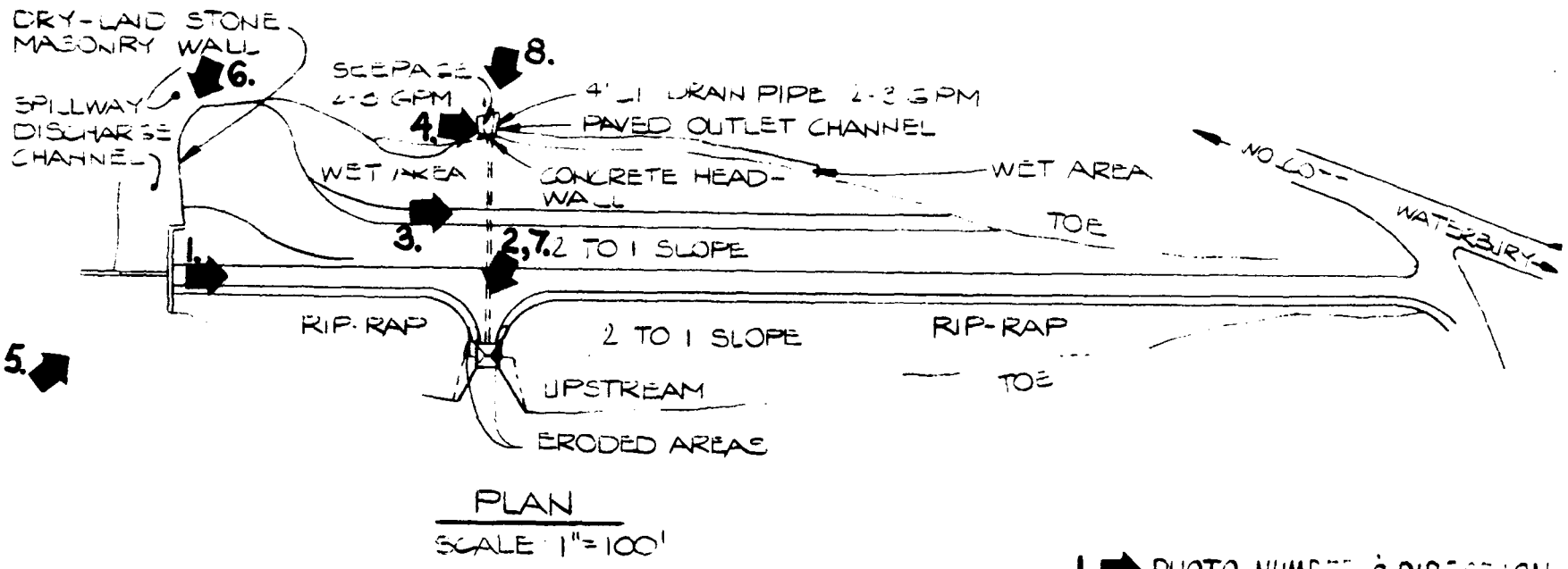
	Drainage Area		Water Surface Area	
Cedar Swamp Pond	0.9	Sq. Miles	130 ±	Acres
Scovill Reservoir (Upper)	7.4	Sq. Miles	115 ±	Acres
Scovill Reservoir (Lower)	0.0	Sq. Miles	5 ±	Acres
Chestnut Hill Reservoir	1.7	Sq. Miles	65 ±	Acres
Hitchcock Lake	0.3	Sq. Miles	100 ±	Acres
Homestead Ave. Dam	7.1	Sq. Miles	6 ±	Acres
Total	17.4	Sq. Miles		

Flood routing studies for the reservoirs were carried out for design floods of 7.5" and 5.1"/ 6 hr storm and the results tabulated below:

	FLOOD ROUTING STUDY							
	SPILLWAY		MAX. WATER SURFACE ABOVE CREST		FREEBOARD		MAX. OUTFLOW Q CFS	
Reservoir	width ft.	depth of crest from top	p=7'5"	p=5'1"	p=7'5"	p=5'1"	p=7'5"	p=5'1"
Cedar Swamp Reservoir	13'-0±	2'-9"	1'-1"	0'-7"	1'-8"	2'-2"	45	2
Scovill Reservoir (upper)	100' & 39'6"	3'-6"	3'-6"	1'-10"	----	1'-8"	3000	1200
Scovill Reservoir (lower)	57' & 79'	3'-10"	3'-4"	1'-10"	0'-6"	2'-0"	2900	1100
Chestnut Hill Lake	28'	4'-8"	2'-9"	1'-5"	1'-11"	3'-3"	450	160
Hitchcock Lake	26'±	2'±	0'-5"	0'-3"	1'-7"	1'-9"	25	10
Homestead Ave. Dam	60' & (40" Sluice gate)	6'-0"	6'-0"	4'-0"	-----	2'-0"	3300	1680

APPENDIX C

DETAIL PHOTOGRAPHS



PLAN

SCALE 1"=100'

1. ➡ PHOTO NUMBER & DIRECTION

PHOTO LOCATION PLAN

CHESTNUT HILL DAM

SHEET C-1

Photo 1 - Top of Dam looking from left end (4/22/81).

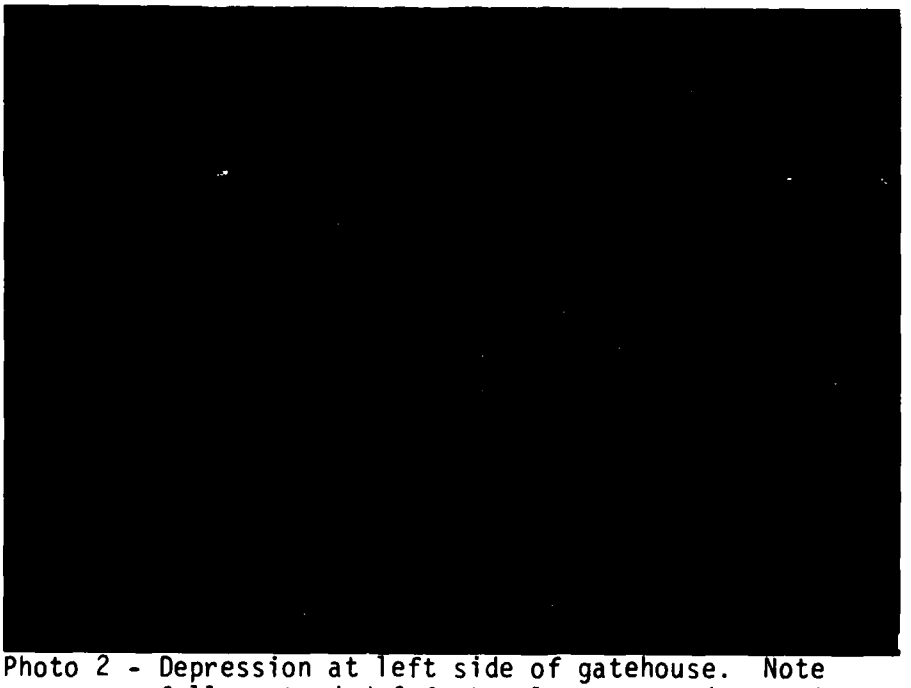


Photo 2 - Depression at left side of gatehouse. Note fully extended 6 foot ruler across depression (4/22/81).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF	Chestnut Hill Dam Old Tannery Brook Wolcott, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN ENGINEER	INSPECTION OF NON-FED. DAMS	CE# 27 785 KI DATE June '81 PAGE C-1

Photo 3 - Downstream slope of dam showing berm of right-central portion of photo (4/22/81).

Photo 4 - Four inch cast iron pipe which appears to be a toe drain outlet. Turbidity in water was caused by inspection team, not by flow from pipe (4/22/81).

Photo 5 - Spillway. Note spalling of right training wall and short section of end wall exposed at left of spillway (4/22/81).

Photo 6 - Spillway discharge channel and dry-laid masonry wall. Note trees and debris in channel (4/22/81).

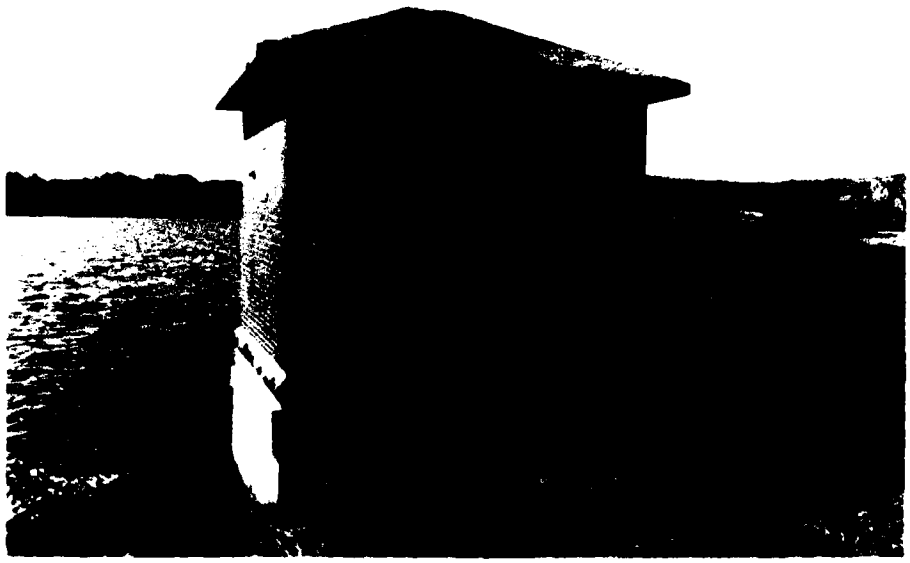
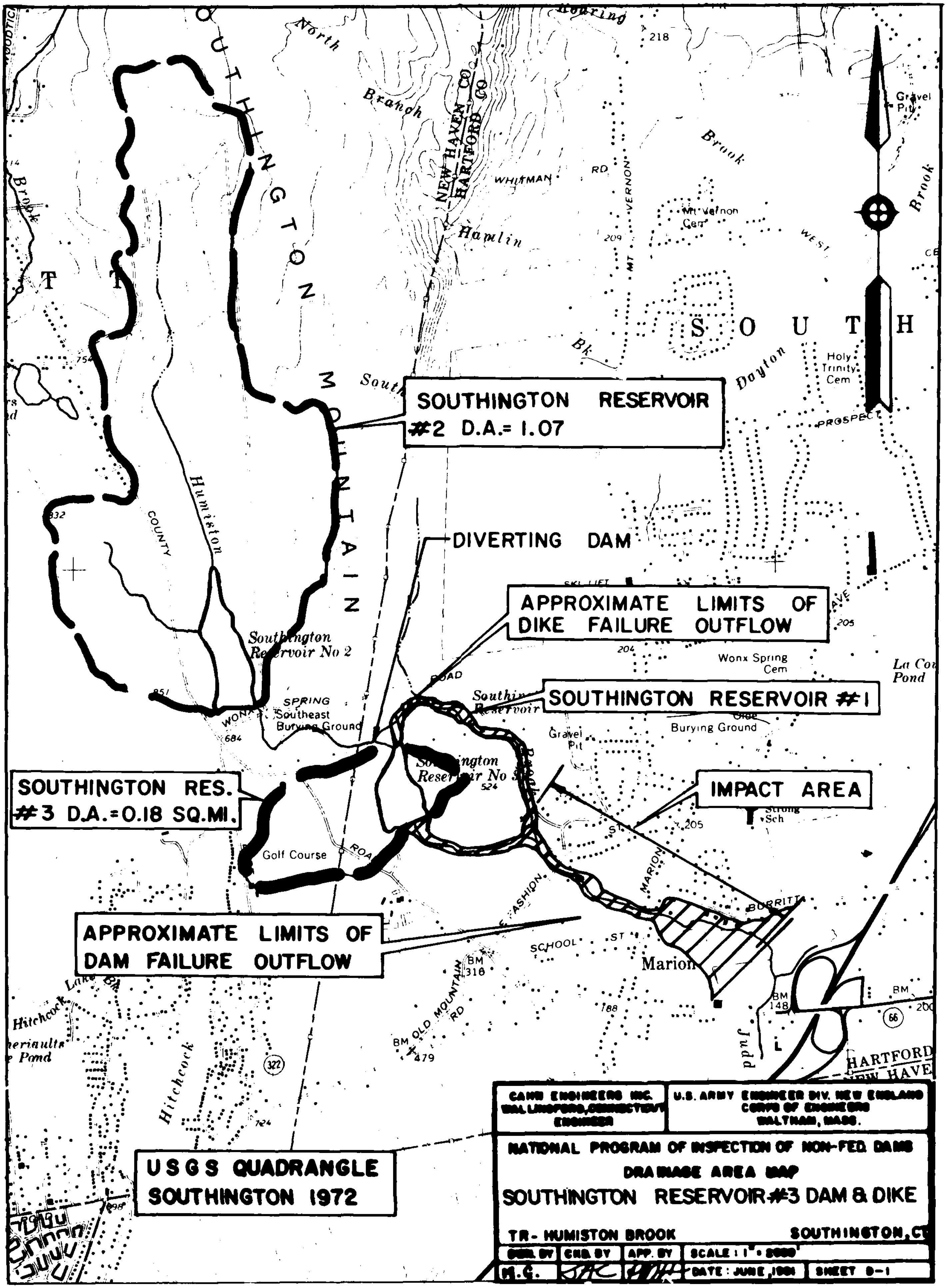


Photo 7 - Gatehouse (4/22/81).

Photo 8 - Low-level outlet headwall. Note spalling of concrete and standing water in outlet channel (4/22/81).

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS


SOUTHINGTON RESERVOIR
#2 D.A.= 1.07
DIVERTING DAM
APPROXIMATE LIMITS OF
DIKE FAILURE OUTFLOW
SOUTHINGTON RESERVOIR #1
IMPACT AREA
SOUTHINGTON RES.
#3 D.A.=0.18 SQ.MI.
APPROXIMATE LIMITS OF
DAM FAILURE OUTFLOW
USGS QUADRANGLE
SOUTHINGTON 1972
SOUTHINGTON MOUNTAIN
Southington Reservoir No 2
Humiston
COUNTY
Golf Course
Marion
Wonx Spring Cem
Burying Ground
Holy Trinity Cem
NEW HAVEN CO
HARTFORD CO
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
DRAINAGE AREA MAP
SOUTHINGTON RESERVOIR #3 DAM & DIKE
TR- HUMISTON BROOK
SOUTHINGTON, CT

HYDROLOGIC / HYDRAULIC INSPECTION

CHESTNUT HILL RESERVOIR DAM, WOLCOTT, CT.

I) PERFORMANCE AT PEAK FLOOD CONDITIONS

1) PROBABLE MAXIMUM FLOOD (PMF).

a) WATERSHED CLASSIFIED AS "ROLLING"

b) WATERSHED AREA: DA = 1.5 sq mi

NOTE: D.A. FROM CONN. D.E.P. BULLETIN No 1, 1972 (GAZETTEER OF NATURAL DRAINAGE AREAS) p. 64.

c) PEAK FLOWS (FROM NED-ACE GUIDELINES - GUIDE CURVES FOR PMF)

i) FROM GUIDE CURVES BY EXTRAPOLATION TO D.A < 2 sq mi

CSM ≅ 2200 cfs/sq mi

ii) PMF ≅ 2200 × 1.5 = 3300 cfs

iii) ½ PMF = 1650 cfs

2) SURCHARGE AT PEAK INFLOWS (PMF AND ½ PMF)

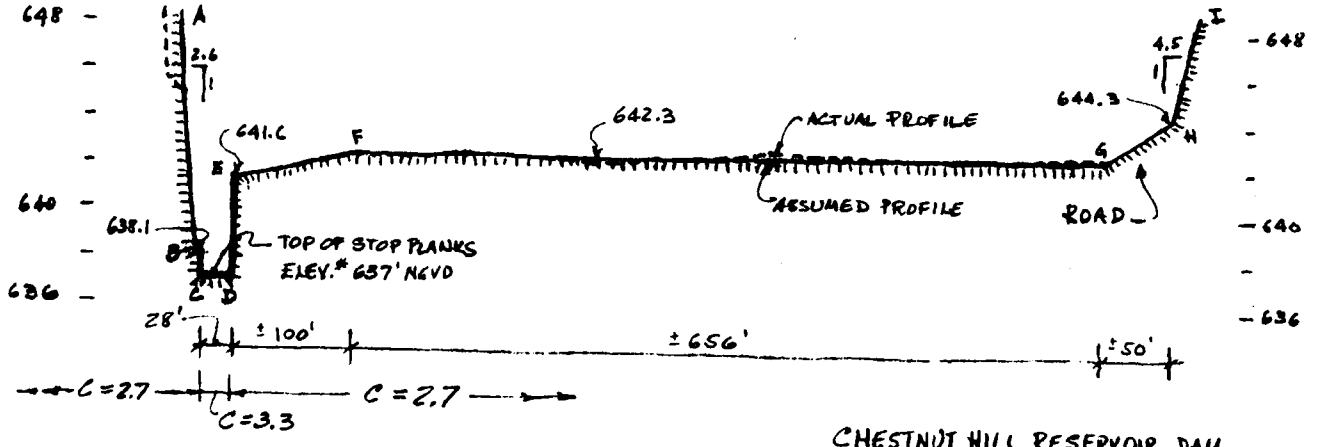
a) OUTFLOW RATING CURVE

c) SPILLWAY AND OUTFLOW PROFILE OF DAM:

SPILLWAY (±) 28' LONG. CONCRETE SPWY WITH 1.5' HIGH STOP PLANKS. SPWY APPROACH CHANNEL SLOPING GENTLY TO WEST OF SPWY. NO TRAINING WALL TO THE LEFT WHERE A (±) 2.5' HIGH × (±) 3.8' LONG CONCRETE ABUTMENT ENDS AT THE GRASSED SIDE SLOPE OF THE OUTFLOW CHANNEL. THEREFORE, THIS SIDE CAN BE CONSIDERED AS AN UNPAVED EMERGENCY SPILLWAY AREA. THE

TOP OF THE STOP PLANKS IS AT ELEV. 637' NGVD*. THE CONCRETE TRAINING WALL TO THE RIGHT OF THE SPWY. ENDS THE DAM EARTH EMBANKMENT (@ TOP ELEV. 642.3' NGVD). A PAVED ROAD INTERSECTION AND NATURAL GROUND AT (±) 4.5H TO 1V SLOPE CLOSES THE OVERFLOW PROFILE (SEE SKETCH BELOW) TO THE RIGHT.

ASSUME C=3.3 FOR THE SPILLWAY DISCHARGE OVER THE STOP PLANKS AND C=2.7 FOR THE DAM AND THE ADJACENT TERRAIN.



NOTE: DATA FROM C.E. OBSERVATIONS ON 2/10/81 BY HLL & R.L.

CHESTNUT HILL RESERVOIR DAM
APPROXIMATE OVERFLOW PROFILE

(ii) THEREFORE, THE OVERFLOW RATING CURVE FOR SURCHARGES (H) ABOVE THE STOP PLANKS (ASSUMED PERMANENT) CAN BE APPROXIMATED AS FOLLOWS:

1') SECTION AB (SLOPED LEFT SIDE OF SPILLWAY):

$$Q_{as} = {}^{**}Q_{AB} = 0.4 \times 2.6 \times 2.7 (H-1.1)^{3/2} = \underline{2.81 (H-1.1)^{3/2}}$$

** SEE NOTE p. D-3

2') SPILLWAY (SECTION CD)

$$Q_s = Q_{CD} = 3.3 \times 28\ H^{3/2} = \underline{92.4\ H^{3/2}}$$

*NOTE: W.S. ELEVATION 637' MSL ON THE USGS WATERBURY, CT. QUADRANGLE SHEET (1968, REV. 1972) IS ASSUMED TO BE THE NORMAL POOL ELEVATION (TOP OF STOP PLANKS) ON NATIONAL GEODETIC VERTICAL DATUM (NGVD) AND TO CORRESPOND TO TOP OF STOP PLANKS ELEV. 601.5' ON THE SCOVILL CO. DWG. "CHESTNUT HILL RESERVOIR-RECONSTRUCTION OF DAM" DATED FEB. 1918.

3') SECTION EF:

$$*(Q_{EF})_1 = 0.4 \times {}^{100}/_{0.7} \times 2.7 (H-4.6)^{3/2} \approx \underline{154 (H-4.6)^{3/2}} \quad H \leq 5.3'$$

$$(Q_{EF})_2 = \underline{\underline{154 [(H-4.6)^{3/2} - (H-5.3)^{3/2}]}} \quad H > 5.3'$$

4') SECTION FG:

$$Q_{FG} = 2.7 \times 656 (H-5.3)^{3/2} \approx \underline{1770 (H-5.3)^{3/2}}$$

5') SECTION GH:

$$*(Q_{GH})_1 = 0.4 \times 25 \times 2.7 (H-5.3)^{3/2} = \underline{27 (H-5.3)^{3/2}} \quad H \leq 7.3'$$

$$(Q_{GH})_2 = \underline{\underline{27 [(H-5.3)^{3/2} - (H-7.3)^{3/2}]}}$$

6') SECTION HI:

$$*Q_{HI} = 0.4 \times 4.5 \times 2.7 (H-7.3)^{3/2} = \underline{4.86 (H-7.3)^{3/2}}$$

THE TOTAL OVERFLOW IS APPROXIMATED BY THE SUM OF ALL THE APPLICABLE FORMULAE ON ITEMS (1') THRU (6'):

$$Q = 92.4 H^{3/2} + 1770 (H-5.3)^{3/2} + 2.81 (H-1.1)^{3/2} + 154 (H-4.6)^{3/2} - 127 (H-5.3)^{3/2} - 22.1 (H-7.3)^{3/2}$$

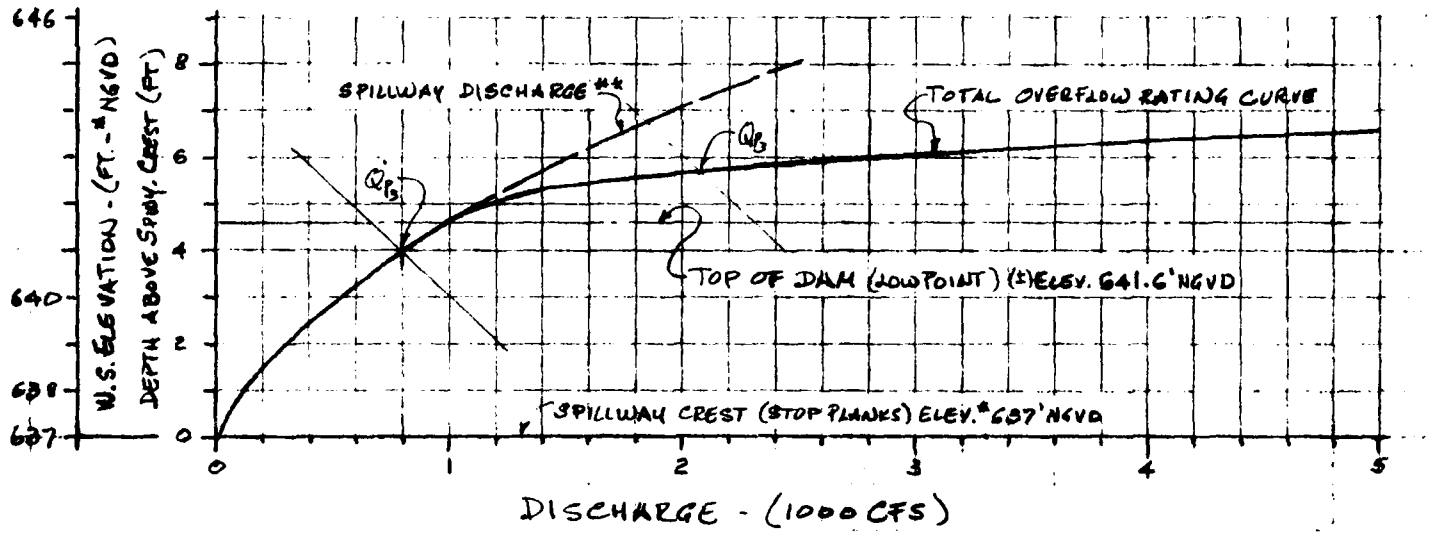
THE CORRESPONDING OVERFLOW RATING CURVE IS PLOTTED ON P. D-4.

* NOTE: FLOW OVER SLOPED SECTIONS BY APPLICATION OF FORMULA GIVEN BY THE USGS ON "MEASUREMENT OF PEAK DISCHARGE AT DAMS BY INDIRECT METHOD" BY H. HULSING (APPLICATIONS OF HYDRAULICS):

$$Q = \frac{2cb}{5(h_b - h_a)} \left[h_b^{5/2} - h_a^{5/2} \right]$$

WHERE Q = DISCH.; C = DISCH COEFFICIENT; h_a & h_b = STATIC HEAD REFERRED TO HIGH AND LOW ENDS OF WEIR, RESPECTIVELY.

ii) CHESTNUT HILL RESERVOIR DAM - OUTFLOW RATING CURVE.



*SEE NOTE p. D-2

**SPILLWAY DISCHARGE INCLUDES DISCHARGE OVER UNPAVED LEFT SIDE SLOPE OF SPILLWAY (SEE p. D-1, SECT. 2ai). THIS ADDITIONAL OVERFLOW IS GIVEN BY: $Q_{OS} = 2.81(H-1.1)^{5/2}$; (p. D-2) AND CORRESPONDS TO (±) 6% OF Q_{P_3} AND (±) 5% OF Q'_{P_3}, RESPECTIVELY.

b) SURCHARGE DEPTHS TO PASS PEAK INFLOWS ($Q_{P_1} \& Q'_{P_1}$)

i) @ $Q_{P_1} \cong$ PMF = 3300 CFS $\quad H_1 \cong \underline{6.1'}$

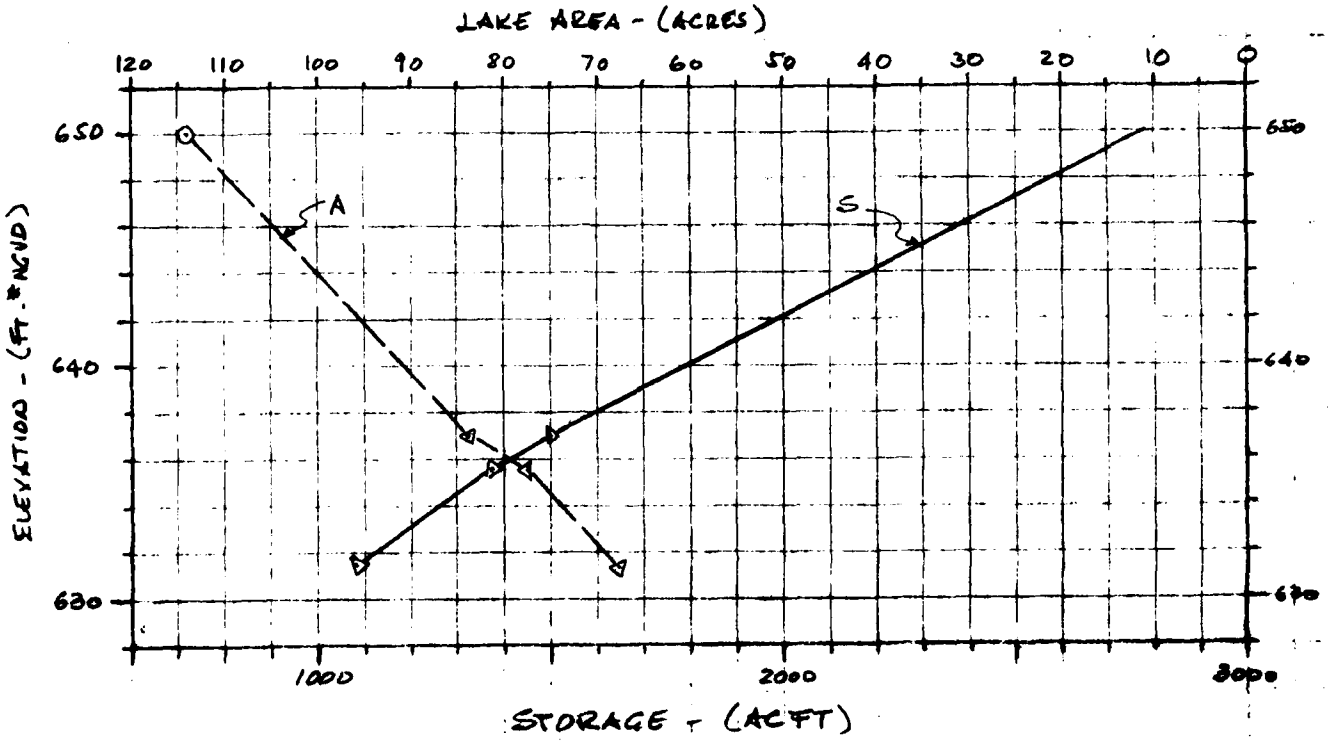
ii) @ $Q'_{P_1} \cong \frac{1}{2}$ PMF = 1650 CFS $\quad H'_1 \cong \underline{5.5'}$

c) EFFECT OF SURCHARGE - PEAK OUTFLOWS.

i) ASSUME NORMAL POOL AT TOP OF STOP PLANKS ELEV. 637' NGVD (p. D-2)

ii) WATERSHED D.A. = 1.5 SQ MI (SEE p. D-1)

iii) LAKE AREA / STORAGE RATING CURVES - CHESTNUT HILL RESERV. DAM:



Δ DATA FROM SCOVILL CO. DRAWING "CHESTNUT HILL RESERVOIR - RECONSTRUCTION OF DAM" DATED FEB. 1918

⊙ AREA MEASURED ON USGS WATERBURY, CT. QUAD. SHEET (1968 / REV. 1972)

* SEE p. D-2

iv) PEAK OUTFLOWS® (Q_{P_3} & Q'_{P_3})

$Q_{P_3} \cong 2060^{CFS}$ $H_3 \cong 5.7'$ (ELEV. 642.7' NGVD)

$Q'_{P_3} \cong 780^{CFS}$ $H'_3 \cong 4.0'$ (ELEV. 641.0' NGVD)

® DETERMINED AS IN THE FOLLOWING EXAMPLE BY THE APPROX. ROUTING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD AND 19° MAX. PROB. P.O. IN NEW ENGLAND.

1) FOR HYPOTETICAL SURCHARGES: $H_a = 8.0'$ AND $H_b = 4.0'$:

$V_a = 792^{AC FT}$; $S_a = 9.91"$; $(Q_{P_2})_a \cong 1580^{CFS}$ $Q_{P_2} = Q_{P_1}\left(1 - \frac{S}{19}\right)$

$V_b = 396^{AC FT}$; $S_b = 4.95"$; $(Q_{P_2})_b \cong 2440^{CFS}$

2) INTERSECT OF LINE $(Q_{P_2})_a$ W/RATING CURVE (P. D-4) DETERMINES Q_{P_3} AND H_3

3) SPILLWAY CAPACITY RATIO TO PEAK CONDITIONS:

SPILLWAY CAPACITY TO:	SURCH* H (FT)	W.S. ELEV. (FT-NGVD)	SPILLWAY CAPACITY** (CFS)	SPILLWAY CAPACITY AS % OF PEAK OUTFLOWS	
				Q_B (2060 CFS)	Q'_B (780 CFS)
½ PMF	4.0	641.0	780	—	100
TOP OF DAM ◉	4.6	641.6	980	48	126
PMF	5.7	642.7	1380	67	—

*SURCHARGE ABOVE SPILLWAY CREST (STOP PLANKS) - ELEV. 637' NGVD
** INCLUDES DISCHARGE OVER UNPAVED LEFT SIDE SLOPE OF SPILLWAY (SEE NOTE p. D-4)
◉ FIRST POINT OF OVERTOPPING (ELEV. OF RIGHT SPILLWAY HEADWALL)

CHESTNUT HILL RESERVOIR DAM

II) DOWNSTREAM FAILURE HAZARD

1) POTENTIAL IMPACT AREA

MANY HOUSES OF WHICH AT LEAST FIVE HAVE FIRST FLOOR ELEVATIONS BETWEEN (±) 8' AND 13' ABOVE THE STREAM; APARTMENT COMPLEXES WITH F.F. ELEVATIONS BETWEEN ±10' AND 12'; A LARGE SHOPPING MALL (FF ±13') AND SEVERAL FACTORIES WITH FIRST FLOOR ELEVATIONS BETWEEN ±6' AND 14' ABOVE THE STREAMBED ARE LOCATED WITHIN 7000' TO 14000' FROM CHESTNUT HILL RESERVOIR DAM, ALONG OLD TANNERY BROOK AND MAD RIVER. THEREFORE, THIS AREA IS CONSIDERED THE POTENTIAL INITIAL IMPACT AREA IN CASE OF FAILURE OF CHESTNUT HILL RESERVOIR DAM. OTHER POTENTIAL IMPACT AREAS ARE LOCATED FURTHER D/S AS MAD RIVER ENTERS DENSELY POPULATED SECTIONS OF THE CITY OF WATERBURY.

2) FAILURE AT CHESTNUT HILL RESERVOIR DAM:

ASSUME SURCHARGE TO FIRST POINT OF OVERTOPPING OF THE DAM, ELEV. 642.6' NGVD.

a) HEIGHT OF DAM*: $H_{max} \cong 36.8'$ (TOP OF DAM ± ELEV. 642.3' NGVD - OUTLET PIPE CHANNEL ± ELEV. 605.5' NGVD)

b) MID-HEIGHT LENGTH*: $\ell \cong 445'$

c) BREACH WIDTH (SEE NED-ACE D/S DAM FAILURE GUIDELINES)

$W = 0.4 \times 445' = 178'$ ∴ ASSUME $W_b \cong \underline{170'}$

d) ASSUMED WATER DEPTH AT TIME OF FAILURE: $Y_o = 36.1'$

e) SPILLWAY DISCHARGE AT TIME OF FAILURE $Q_s = 980^{cfs}$ (SEE p. D-6)

* FROM C.E. MEASUREMENTS ON 2/10/81 BY HLH & RC.

f) BREACH OUTFLOW (SEENED-ACE GUIDELINES)

$$Q_b = \frac{8}{27} W_b \sqrt{g}\, y_0^{3/2} \approx 62000^{cfs}$$

g) PEAK FAILURE OUTFLOW (Q_{P_1})

$$Q_{P_1} = Q_s + Q_b \approx \underline{63000}^{cfs}$$

3) FLOOD DEPTH** IMMEDIATELY D/S FROM DAM:

$$y \approx 0.44\, y_0 \approx \underline{15.9'}$$ ** (FROM RETREIVING WAVE TODAY)

4) ESTIMATE OF D/S FAILURE CONDITIONS AT POTENTIAL IMPACT AREA.

a) THE OLD TANNERY BROOK CHANNEL D/S FROM CHESTNUT HILL RESERVOIR DAM IS APPROXIMATELY V-SHAPED WITH (±) 15" AND 10" TO 1' SIDE SLOPES AND A REACH SLOPE OF (±) 1.4%. THE MAD RIVER CHANNEL D/S FROM OLD TANNERY BROOK IS GENERALLY TRAPEZOIDAL IN CROSS SECTION WITH (±) 50' BASE, (-) 30" TO 1' SIDE SLOPES AND A REACH SLOPE OF (±) 0.17% (ASSUME $n = 0.050$ FOR THE BROOK AND RIVER CHANNELS AT FLOOD STAGE).

b) RESERVOIR STORAGE AT TIME OF FAILURE

$$S_{CAL.C} \approx 1900^{AC\text{-}FT} \quad \text{(SEE P. D-5)} \qquad S/2 \approx 950^{AC\text{-}FT}$$

c) APPROXIMATE STAGE AT POTENTIAL IMPACT AREA

i) 1ST REACH, (±) 7000' TO A SMALL POND ON OLD TANNERY BROOK

$Q_{P_1} = 63000^{cfs}$; $y_1 \approx 18.1'$; $V_1 = 662^{AC\text{-}FT}$; $Q_{S_2} = 41100^{cfs}$; $y_2 \approx 15.5'$
$V_2 = 480^{AC\text{-}FT}$; $\bar{V} = 571^{AC\text{-}FT}$; $Q_{S_3} = \underline{44100}$; $y_3 \approx \underline{15.9'}$

ii) 2ND REACH, (±)5000' TO MAD RIVER

$Q_{P_1} \cong 44100^{cfs}$; $Y_1 \cong 15.9'$; $V_1 = 361^{ACFT}$; $Q_{P_2} = 35700^{cfs}$; $Y_2 \cong 14.7'$
$V_2 = 309^{ACFT}$; $\bar{V} = 335^{ACFT}$; $Q_{P_3} = \underline{36400}^{cfs}$; $Y_3 \cong \underline{14.8}'$

iii) 3RD REACH, (±) 2000' ON MAD RIVER TO SHOPPING MALL.

ROUTING THE DAM FAILURE FLOOD ALONG MAD RIVER YIELDS AT THE SHOPPING MALL AREA A FLOOD $(Q_{P_3})_{MR} \cong 38300^{cfs}$; $(Y_3)_{MR} = 13.9'$.

d) APPROXIMATE STAGE BEFORE FAILURE:

$Q_S = 980^{cfs}$ (SEE P.P. D-6 & D-8) $\therefore$ $Y_S \cong \underline{3.8}'$ (ASSUMED (±) THE SAME FOR BOTH, THE OLD TANNERY BR. & MAD RIVER)

e) RAISE IN STAGE % FROM CHESTNUT HILL RESERVOIR DAM.

DEPENDING ON THE LOCATION ALONG THE OLD TANNERY BROOK REACH DESCRIBED AS THE POTENTIAL INITIAL IMPACT AREA (P. D-7), THE RAISE IN STAGE UPON FAILURE OF CHESTNUT HILL RES. DAM IS ESTIMATED TO BE BETWEEN (±)11' AND 12' AND, AT MAD RIVER, OF (±) 10'.

CHESTNUT HILL RESERVOIR DAM

III) SELECTION OF TEST FLOOD

1) CLASSIFICATION OF DAM ACCORDING TO NED-ACE GUIDELINES:

a) SIZE: *STORAGE (MAX) $\simeq$ 2000 AC-FT (1000 < S < 50000 AC-FT)
*HEIGHT (MAX) $\simeq$ 36.8' (25 < H < 40 FT)

*STORAGE: SEE p. D-5 ; HEIGHT: SEE p. D-7

∴ SIZE CLASSIFICATION: INTERMEDIATE

b) HAZARD POTENTIAL: AS A RESULT OF THE P/S FAILURE ANALYSIS AND IN VIEW OF THE IMPACT THAT FAILURE OF CHESTNUT HILL RESERVOIR DAM MAY HAVE ON THE POTENTIAL IMPACT AREA (P. D-7), THE DAM IS CLASSIFIED AS HAVING:

HAZARD CLASSIFICATION: HIGH

2) TEST FLOOD: PMF = 3300 CFS

THIS SELECTION IS BASED ON THE RESULTS OF THE PREVIOUS ANALYSIS AND CLASSIFICATION.

CHESTNUT HILL RESERVOIR DAM

IV) SUMMARY

1) TEST FLOOD = PMF = 3300 CFS
(PARALLEL COMPUTATIONS HAVE BEEN MADE FOR ½ PMF ≅ 1650 CFS AND ARE ALSO SUMMARIZED BELOW)

2) PERFORMANCE AT PEAK FLOOD CONDITIONS:

a) PEAK INFLOWS: Q_{P_1} = PMF ≅ 3300 CFS Q'_{P_1} = ½PMF ≅ 1650 CFS
b) PEAK OUTFLOWS: Q_{P_3} ≅ 2060 CFS Q'_{P_3} ≅ 780 CFS
c) SPILLWAY CAPACITY: (SEE TABLE p. D-6)
d) PERFORMANCE:
i) AT TEST FLOOD: DAM OVERTOPPED (±) 1.1' (W.S. ELEV. 642.7' NGVD)
ii) AT ½ PMF: FREEBOARD (±) 0.6' (W.S. ELEV. 641.0' NGVD)

3) DOWNSTREAM FAILURE CONDITIONS:

a) PEAK FAILURE OUTFLOW: Q_{P_1} = 63000 CFS
b) FLOOD DEPTH IMMEDIATELY D/S FROM DAM: Y_0 ≅ 15.9'
c) CONDITIONS AT INITIAL IMPACT AREA (±) 7000' D/S FROM DAM:
i) STAGE BEFORE FAILURE: Y_S ≅ 3.8' (Q_S ≅ 980 CFS)
ii) STAGE AFTER FAILURE: Y_3 ≅ 15.9' (Q_{P_3} ≅ 44100 CFS)
iii) RAISE IN STAGE AFTER FAILURE: ΔY ≅ 12.1'
d) CONDITIONS AT OLD TANNERY BRK. JUNCTION WITH MILL RIVER (±) 12000' FROM DAM:
i) STAGE BEFORE FAILURE: Y_S ≅ 3.8' (Q_S ≅ 980 CFS)
ii) STAGE AFTER FAILURE: Y_3 ≅ 14.8' (Q_{P_3} ≅ 36400 CFS)
iii) RAISE IN STAGE AFTER FAILURE: ΔY ≅ 11.0'

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

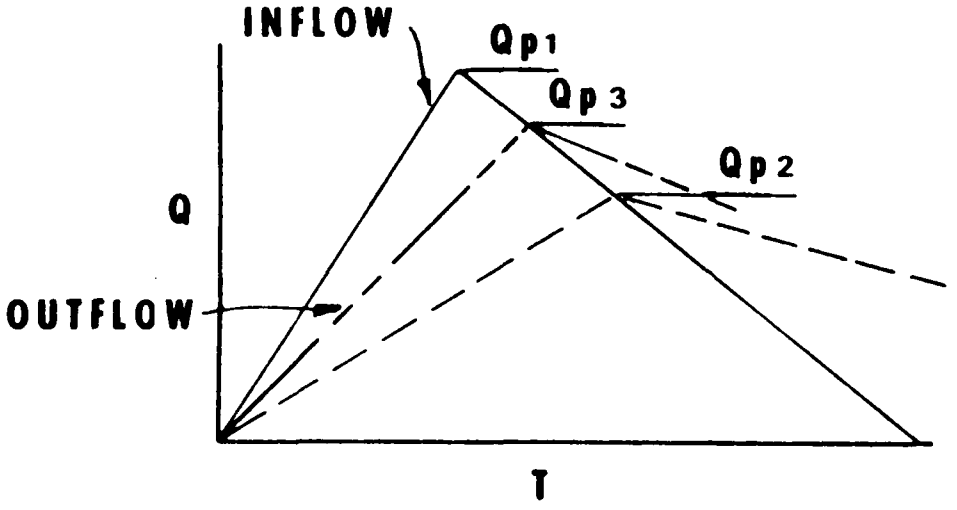
MAXIMJM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD (Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Qp_1) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Qp_1".

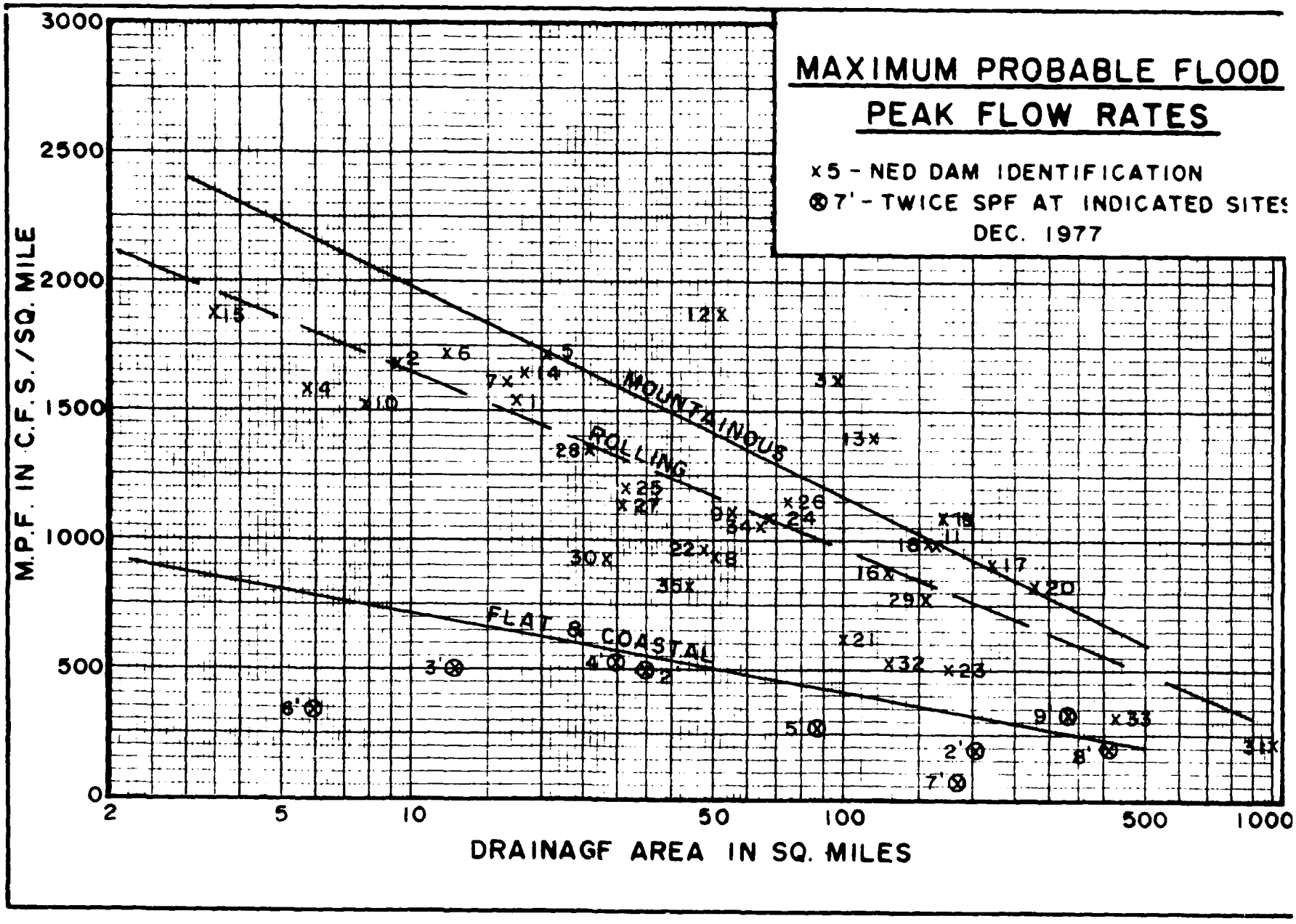
b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore:

$$Qp_2 = Qp_1 \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Qp_3".


MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGF AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000
MOUNTAINOUS
ROLLING
FLAT & COASTAL
X15
X4
X10
X2
X6
7X
X14
X1
X5
12X
3X
13X
28X
X25
X27
X26
9X
34X
X24
X18
X11
19X
X17
X20
16X
29X
30X
22X
X8
35X
X21
X32
X23
X33
31X
3'⊗
4'⊗
⊗2
6'⊗
5⊗
9⊗
2'⊗
8'⊗
7'⊗

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

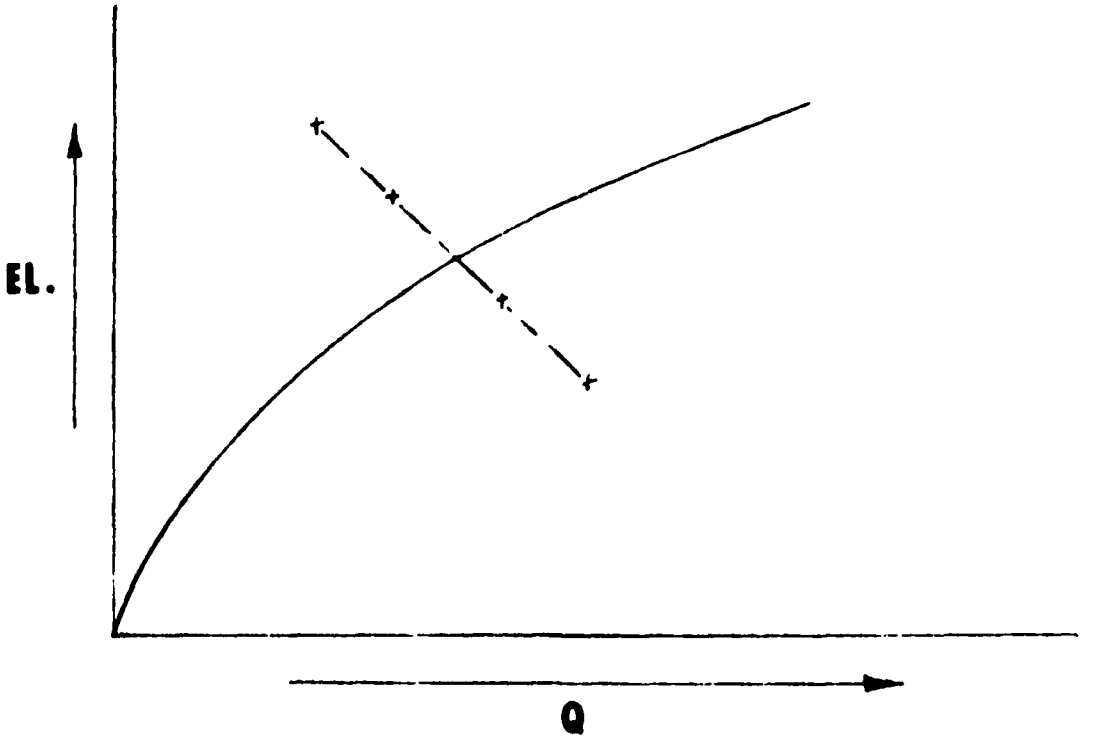
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

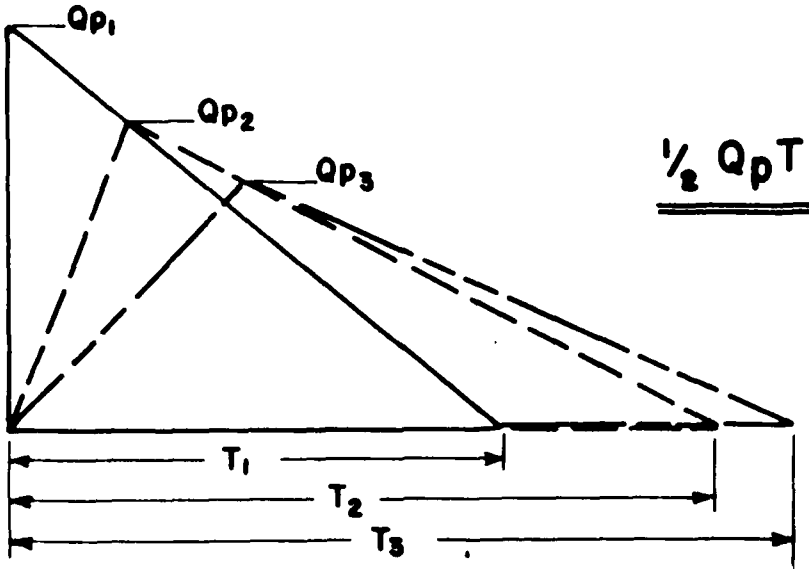
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$\frac{1}{2} Q_p T = 12 S$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g} Y_o^{\frac{3}{2}}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(TRIAL) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{AVG}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME

FILMED

DTIC